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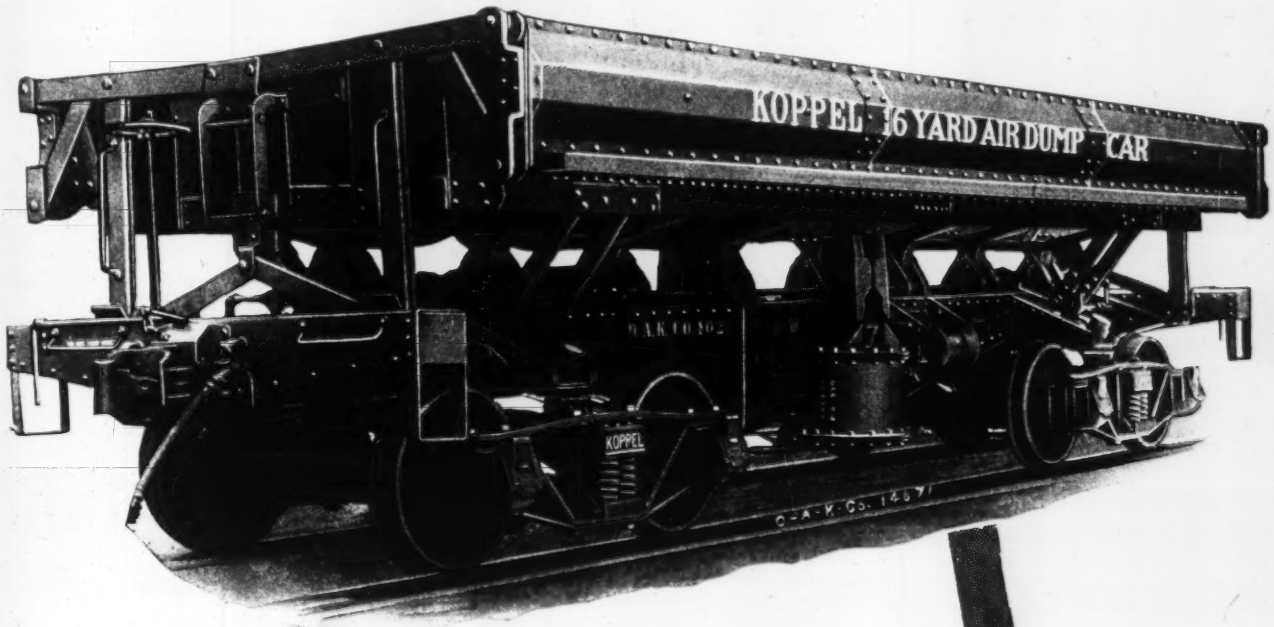

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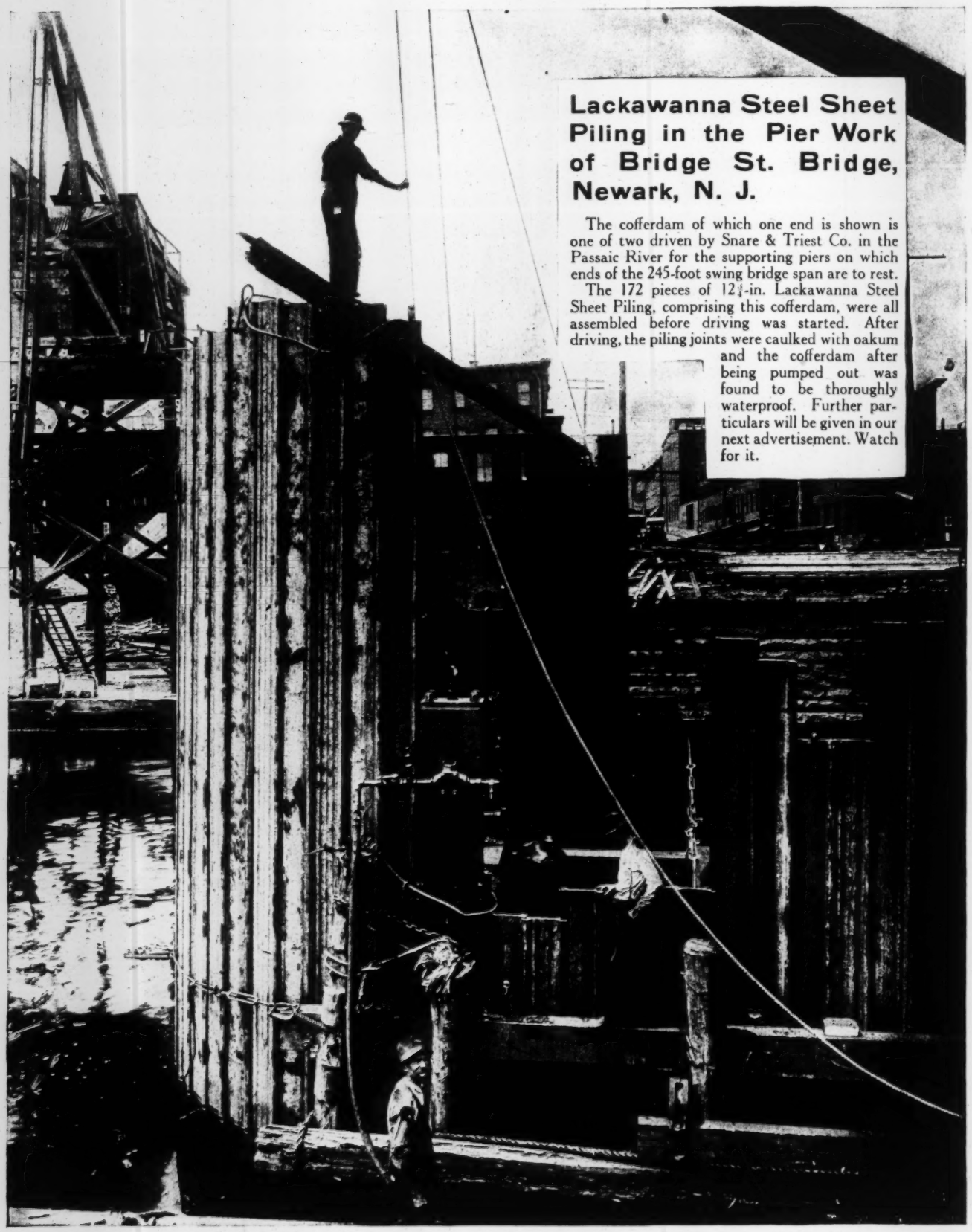

Lackawanna Steel Sheet Piling in the Pier Work of Bridge St. Bridge, Newark, N. J.

The cofferdam of which one end is shown is one of two driven by Snare & Triest Co. in the Passaic River for the supporting piers on which ends of the 245-foot swing bridge span are to rest.

The 172 pieces of 12¼-in. Lackawanna Steel Sheet Piling, comprising this cofferdam, were all assembled before driving was started. After driving, the piling joints were caulked with oakum and the cofferdam after being pumped out was found to be thoroughly waterproof. Further particulars will be given in our next advertisement. Watch for it.

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Engineering News

CHARLES WHITING BAKER, *Editor*

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What The Buyer Needs To Know

Number Thirteen

All mankind is divided into three classes, namely and to wit:

Those who say, "Let's do something new;" those who say, "It can't be done;" and those who will not say anything. The latter, we believe, are popularly supposed to have the "judicial" mind. They are charter members of the "I-Told-You-So-Club," and hold all the offices in the Gloom Union and the Gloaters' Federation.

This has accounted for whatever progress we've made, for whatever backward steps we've taken and for most of our "stand pat" periods.

These very human traits, taken in connection with the history of invention, are interesting.

Some learned gentlemen, we believe, have promulgated the theory that invention comes in response to demand. Viewed in the light of the history of most prominent inventions, the theory is about as sound as the one of perpetual motion.

Nothing *new* is ever *bought*; it has to be *sold*.

Great inventions are brought forward —and nobody wants them. A dollar in real money is preferable apparently to stock in the concern that is to manufacture the article and a plugged quarter looks better than an investment in the article itself.

Indeed, the inventor not only has to fight indifference—he is often up against violent antipathy.

Women cried when cook stoves were adopted; men fought when textile machinery was introduced; a Boston mob smashed the first sewing machine that was exhibited by Howe.

It took McCormick fourteen years before he sold his first hundred reaping machines. Bell fought opposition left, right and center to get the telephone financed. Field laid the Atlantic cable amidst the jeers and hoots of all nations.

This condition brought into existence a comparatively new type of specialist, the highly trained salesman.

In the "good old days" the inventor was his own manufacturer and his own salesman. Being an inventor he was just naturally a dismal failure at the other two things. For your inventor is a self-centered individual who needs a guide outside his own particular domain.

The trinity, then, is the inventor, the manufacturer and the salesman. The one to conceive, the next to make economically and well, the last to sell, *at a profit*.

The development of the highly trained salesman is a boon to civilization. He's the forerunner of prosperity and the enemy of stagnation! He's the old original self-starter and runs on ball bearings.

The occasional bore, the smart Aleck and the over persistent are mere masqueraders—they don't really belong to the elect.

For the real salesman is never any of those things, any more than an expert at Kelly pool is a billiardist.

Running hand in hand with the modern salesman is a still newer specialty—advertising. The purpose and aim of each is similar. It is to make known what the inventor has devised and the manufacturer made and to sell the product at a profit so that the whole combination may be labeled "Success."

In this good work each has a distinct field of action and neither need tread on the other's toes. The work of informing can be done more quickly and more cheaply by advertising. The work of selling can seldom be done by any other than the salesman—very, very seldom indeed in machinery and kindred lines.

And so wise manufacturers of today are using both methods because they can market their goods more cheaply and more surely than when they attempt to get along with one of them alone.

When this truth reaches home, wise buyers will insist on advertised goods in preference to others. It's a pretty safe gamble that the economical selling method will reflect in the lowered price of the product.

Engineering News

VOLUME 69 MARCH 27, 1913 NUMBER 13

A 5350-hp. Steam-Turbine Generator Unit with Speed-reduction Gears

The two largest direct-current generators yet built have recently been installed in the Canal Road plant of the Cleveland (Ohio) Electric Illuminating Co. Each of these is a 3750-kw. 275-volt 180-r.p.m. shunt-wound machine driven by a Parsons-type steam turbine through Westinghouse reduction gears having a 10:1 ratio. These machines are intended to supply the light and power circuits in the business section of this city. The company's installation was originally a direct-current three-wire system, but, with increase of business, alternating-current apparatus was used for distribution to outlying districts. The capacity of 200,000 kw. This plant is 3½ miles from the business center of the city, and it was not proposed to abandon the old plant entirely, but rather to convert this latter into a non-condensing direct-current generating station with the exhaust steam from the units here used for the heating mains of the business district.

The size of the equipment noted has been chosen on account of the exhaust steam needed, rather than on account of generator capacity. The machines will be run only during the months in which exhaust steam is required; in the other seasons rotary converters will be used, drawing power from the alternating-current lines.

The gear in the reduction mechanism (Fig. 2) is 100

A 5350-HP. STEAM TURBINE DRIVING A 3750-KW., DIRECT-CURRENT GENERATOR THROUGH REDUCTION GEAR
(In the Canal Road Plant of the Cleveland Electric Illuminating Co.)

supply system for the congested business district, however, was never changed and up to now for this service about 8000 kw. in engine-driven direct-current generators have been retained. On account of growth of business and poor quality of condensing water obtained from the Cuyahoga River, a new alterating-current station with 22,000 kw. of steam-turbine generators, was built on the lake front, with provisions for an ultimate

*From information furnished by the Westinghouse companies' Publicity Department, East Pittsburgh, Penn.

in. in diameter with 259 helical teeth. The pinion has 26 teeth, giving a ratio of 26 to 259 and introducing the so called "hunting tooth" to prevent frequent recurrence of contact between the same pair of teeth. The full rated load of the gear is 5350 b.hp. and, on account of overloads, this has been increased to 6000 b.hp. without indications of distress. The generator, shown in Fig. 1, is 21 ft. 4 in. in diameter, across the frame-feet base, and it stands 14 ft. 2 in. above the floor. The entire unit is 37 ft. 2 in. long and this length is divided be-

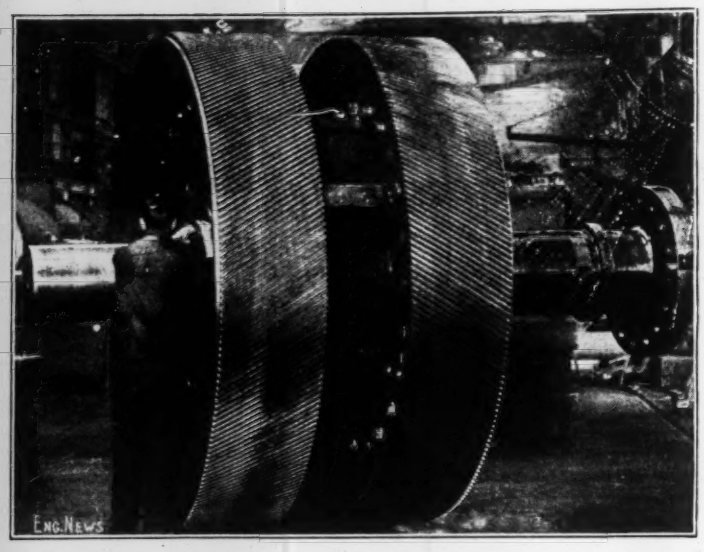


FIG. 2. DOUBLE HELICAL GEAR OF CLEVELAND GENERATING UNITS

tween the generator, gear, and turbine as 10 ft. 1 in., 12 ft. 5 in., and 14 ft. 8 in., respectively.

The turbine and gear were made by the Westinghouse Machine Co., and the generator by the Westinghouse Electric & Manufacturing Co., Pittsburgh.

Building a Paved Roadway across a Swamp*

BY JAMES OWEN†

The recent construction of the Plank Road from Newark to Jersey City, N. J., furnished some interesting foundation problems. The road was constructed partly on an old causeway that was built in Colonial times and partly on a new fill in the territory known as the Jersey meadows. To properly appreciate the conditions here it must be remembered that what is known as the Jersey meadows was originally an open estuary with the tide ebbing and flowing over it. By gradual deposition, and with wash from the highlands, this estuary was gradually filled up with material more or less solid in consistency to a height of about 1 ft. above low tide. When vegetation started in the shape of coarse grass or reeds, this growth accelerated the deposition until the high-water level was reached, when, of course, the earth accumulation ceased, but the perennial growth of reeds continued.

The foundation under the Plank Road is consequently soft silt with occasional pockets of clay. Nearer the highlands shallow beds of coarse sand are encountered. The depth of the silt was not ascertained; 80-ft. piles were driven without touching hard bottom. On top of this silt is about 1 ft. of decayed vegetation and from 2 to 4 ft. of matted roots which form the upper surface. The original foundation of the road may therefore be considered as unstable and liable to movement under pressure.

Filling in swamps for roads and railroads is, of course, a frequent occurrence; but, as a rule, with unstable foundations it is possible to relevel or realign the surface either of the road or track. Whatever settling occurs can be relieved by surface work. In the case of the Plank Road such a proposition could not be entertained for it was desired and expected to have a finished surface of block pavement with no settling after the completion of the work; practically to have an unadjustable top.

Two instances of a similar kind came to the author's mind before beginning the work: At the corner of Grand and Henderson Sts., Jersey City, the vibration due to the nearby trolley cars or loaded wagons was very perceptible when standing on the sidewalk, although this section of the old meadows is solidly built up and the streets permanently paved, probably over 50 years ago; in the valley of the Red River of the North in North Dakota, the surface is also soft silt to an unknown depth, covered with a mat of about 6 ft. of solid prairie grass. All buildings erected in that locality were built on this grass mat, never penetrating it. In one case an ambitious architect in erecting a bank building, built it on piles, and a resident told me that this was the only building which ever settled out of place.

The two salient points of construction in the writer's mind were, therefore, to put in enough material to prevent perceptible vibration, and in placing this material care was to be used not to break through the surface. As to the vibration the problem was to find the economical limits of fill for the character of travel, it being understood, of course, that there should be a sufficient mass of fill to take up the momentum of a moving vehicle. For a roadway to carry an 18-ton motor truck or a 35-ton trolley car the mass required would be less than for a 100-ton locomotive.

The original design of a minimum fill of 4 ft., probably haphazard at first inception, resulted in practically eliminating perceptible vibration for the travel to be encountered on the Plank Road, it being understood that this surface fill above the old level of the meadow was augmented by from 60 to 70% more which had sunk below.

The next problem was to make a fill that would not break the surface mat. In the writer's experience with such work in earlier years, and in common with the practice of most other engineers in the country, the fill was dumped on the meadows in a ruthless manner, and sufficient earth was finally brought in to reach the desired grade. The mat was broken through and in many cases swelled up on the side of the fill itself.

To obviate this trouble the specifications were carefully drawn. They may be briefly outlined as follows: None other than good material was allowed and no large stones were used, and whatever stones were applied were scattered. The first layer of fill was not more than 2 ft. thick, which was then rolled with a horse-roller; and each deposition was made in layers of 1 ft. and no more. As soon as conditions were favorable a steam-roller was used and each layer was thoroughly rolled until the final surface was reached. No permanent pavement was laid until the fill had gone through one winter. In some places that were exceptionally soft the horse-roller was omitted and sufficient material was dumped to enable the steam-roller to get to work.

This rolling was sometimes a heart-breaking proposition as continual miring and stalling was experienced. On one section this was obviated by placing a layer of stone screenings from 4 to 6 in. thick. The material used for the fill was good selected earth from New York and Jersey City.

*Based on a paper read before the meeting of the County Engineers of New Jersey, Jan. 31, 1913.

†Consulting Engineer, 196 Market St., Newark, N. J.

The precautions taken in construction have so far proved efficacious. In only one case was the surface crust broken through, and that was for about 150 ft., causing a crack in the fill. This happened in the early part of the work and was soon remedied. In another case where the fill was outside the old causeway a settlement appeared for about 100 ft. after the curb and concrete foundation were laid. The curb was reset to true line and a supplementary surface was put on the concrete. On the whole length of the finished roadway any perceptible vibration has been entirely eliminated.

The road as built is 100 ft. wide with two high-speed trolley tracks in the center elevated 6 in. above the roadways, two roadways 27 ft. wide, one side paved with close grouted granite blocks, and the other side with creosoted wood blocks, and two outside sidewalks 10 ft. wide.

The selection of the pavement was a subject of protracted controversy between the advocates of wood and granite block. A compromise was finally reached with the above result. A curious fact was noted with the opening of travel in that 70% of the travel is on the wood block as against 30% on the granite block. The travel on the road is very large for it is practically the only wagon road from New York City westward to the rest of the United States.

In using the road, in regard to vibration, one curious result was noticed, and that was the difficulty in getting the grouting on the granite to stay. In other contracts for close jointed grouted granite blocks the grouting was placed in the same manner with satisfactory results, waiting the usual period of 10 to 14 days before travel was allowed. On the Plank Road, however, the grouting simply wore off almost immediately. My assistant thought that the vibration of the whole road destroyed the bond. This vibration was, of course, imperceptible to the senses but undoubtedly existed.

This experience recalled two previous experiences of the writer where vibration was a cause of failure. In building an arch bridge of brick along a railroad track the centers of the arch were struck in accordance with the usual practice, but the arch fell in. In another case 50 ft. of a brick sewer, carried along the line of the railroad track, fell in. This question of vibration may, therefore, be important to consider in similar instances.

In conclusion it may be said that the part of the Plank Road now in use (about two-thirds is finished) shows no sign of settlement or deviation and is a satisfactory piece of work.

❖

Fast Work in Driving a Mine Drift

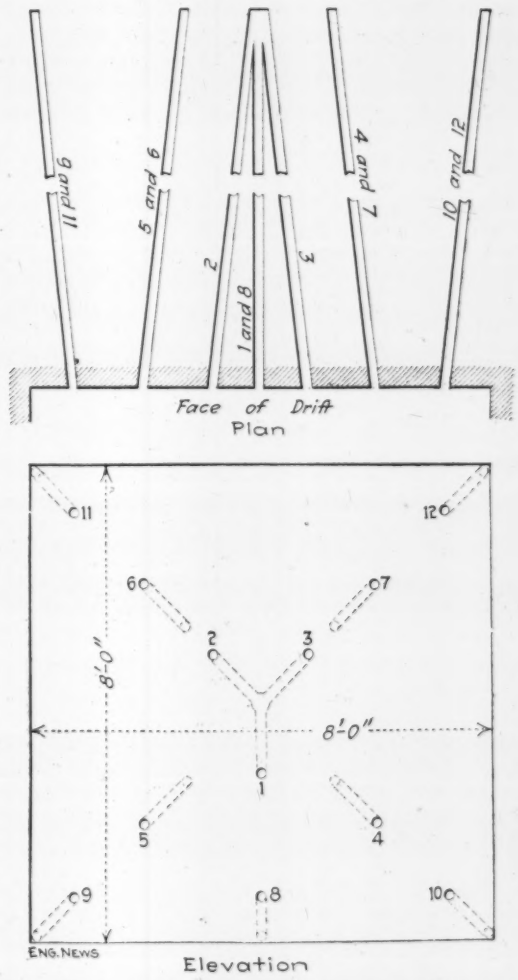
By E. T. Lednum.*

On Feb. 1, 1913, the Vinegar Hill Zinc Co. completed a double-track tramming drift at the Ellsworth Mine, near Livingston, Wis. (in the lead and zinc district). The drift is 8 ft. high and 8 ft. wide, and 391 ft. in length and was driven in 57 shifts of 9 hours each. The total time consumed was 37½ days. Part of the time drilling was carried on during each shift but during the latter part of the work drilling and blasting were done on the night shift only and the muck was removed during the day shift. The drift was in the oil-rock formation of Galena limestone.

*E. I. duPont de Nemours Powder Co., Chicago, Ill.

A part of the drift was driven by drilling but seven holes; three cut holes near the bottom in the middle of the heading, drilled to meet at the points; two bottom corner lifter holes, and two upper corner holes. In mineralized ground twelve holes were drilled, and the arrangement of these holes is shown in the accompanying cut. The cut holes Nos. 1, 2 and 3 were drilled to meet at the points and were, of course, fired first. The average advance per shift was 6.85 ft. The average advance per day was 10.4 ft.

One No. 8 Leyner drill was used, and all holes were drilled 12 ft. deep. The total amount of powder used



Arrangement of Drill Holes for a Mining Drift

was 6930 cartridges (1¼x8-in.) of duPont forcite gelatin and duPont low-freezing gelatin of 40% strength. The powder was exploded with Metallic Cap Mfg. Co. and duPont No. 6 caps; 1012 caps were used and 7613 ft. of Climax-Crescent fuse.

During January the temperature dropped below zero, but at no time did the mining company find it necessary to thaw the duPont new low-freezing gelatin in the thawing magazine. The powder was taken from the storage magazine into the mine at the beginning of the shift and when drilling was finished the powder was ready for use. It was necessary to install a blower, and there was no ventilation. The work was under the direct supervision of Goodsell Billings, Superintendent.

❖

Safety on the Oil-burning Vessels in the Navy will be increased by the provision, on such ships, of an oxygen breathing apparatus. It is estimated that about 10 lives are lost each year on these oil-burning vessels, when men are overcome by gases escaping from the liquid fuel. The apparatus is simple, consisting merely of a small oxygen tank, a flexible tube, and a mouth piece covering the nose and mouth, but not hindering the vision as the usual helmet does.

Stereophotographic Surveying

By Otto Lemberger*

SYNOPSIS—A short history of the development of photographic surveying in Europe, with a few examples of photographic railway location and reconnoissance surveys; the theory of photogrammetry briefly explained; the invention of stereophotogrammetry, its theory and practice. A description of the instruments used in stereophotogrammetry, the stereomicrometer and the stereocomparator with examples of their practical use. A description of the stereoautograph, which automatically draws a map while an operator manipulates the stereocomparator, with examples of its uses and its possibilities.

Although photo-topographical surveys have been made on a large scale in Canada since 1888, and were made by United States government topographers in 1893-94 on the boundary survey between Canada and Alaska, this method of surveying has not gained followers in America outside of the government engineering circles, and is restricted in Canada as well as in the United States to government surveys. American literature is very meager on the subject, and considering the large number of technical schools in this country, it is surprising that more attention is not given to this branch of surveying.

It is one of the objects of this article to show the application of photo-surveying methods to preliminary railway surveys in countries where the customary methods of surveying are very difficult to apply, or fail to give satisfactory results; or where the topographical conditions are such that we can get the quickest and cheapest results by photogrammetry. In the article we will sketch very briefly the history of the method and give only as much theory as is absolutely necessary for understanding the cited examples. For further information the reader is referred to Capt. E. Seville's book "Photographic Surveying," and J. A. Flemer's "Elementary Treatise on Photographic Methods and Instruments,"† in which publications he will find a full treatment of the subject.

Photogrammetry is the art of producing a horizontal, orthogonal projection from a perspective view of a body, i.e., producing topographical maps with contour lines from photographs, based on the laws of perspective and of descriptive geometry. Photogrammetry may be said to be almost as old an art as photography itself. In 1839, when Arago, the famous French scientist, presented to the Chamber of Deputies in Paris the dissertation written by Daguerre and Niepce on the invention of photography, he expressed the opinion that "Photographic pictures are subject to the rules of geometry, and knowing some points, it will be possible to make plans of parts of buildings, which are now inaccessible." He further ventured the opinion, that the time was not far distant when it would be possible to make topographical plans based on photo-topographic surveys. Arago's prophecy was fulfilled a few years later. A French captain, Laussedat by name, was the first to use a camera for topographical surveys.

A method of making a topographical map from a free-hand perspective sketch had been previously devised, and Laussedat applied this method to photography. It was not until the year 1861, however, that Laussedat made the memorable survey of the village of Buc and vicinity, near Versailles, which made his name famous. For this work he received a gold medal in 1863, conferred by the Royal Academy of Science, Madrid. Laussedat died in 1907 at the patriarchal age of 88 years. His thankful pupils, admirers and the French government have erected a monument to his memory in his native town, Moulins, with the following inscription: *Tout à la science, pour son pays.*

*252 W. 121 St., New York City.

†Reviewed in Eng. News, July 12, 1906.

Theory of Photogrammetry

To locate a point by means of photography, we must have this point on two photographs. In Fig. 1, let PP and $P'P'$ represent vertical photographic plates, S_1 and S_2 the two positions corresponding to the nodal points of the objective lenses of the camera; a' and a'' the projections of a point A of the landscape on the two plates. The two elements of a picture are the principal point or eye-point and the distance line. The principal point is

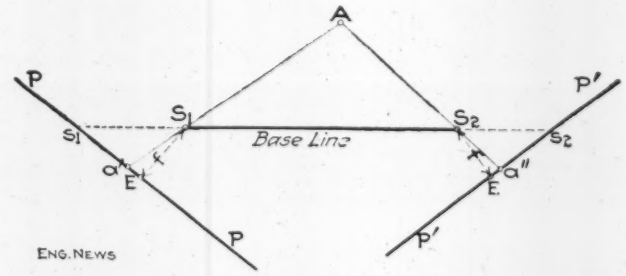


Fig. 1. Fundamental Principle of Locating a Point in Space by Means of Photographs

the point where a perpendicular from the nodal point, or point of view, intersects the plate, marked E in Fig. 1. The length of this perpendicular is the distance line. If the photographs are so taken that they also contain pictures of the points of observation S_1 and S_2, we can locate these points by intersection, based on the principle of the planetable, but having the advantage over the planetable that the problem is solved in the office.

In Fig. 2, P represents the photographic plate in vertical position, S_1 the nodal point or focus of the lens, A a point in space, a its projection or picture. A horizontal plane through S_1 intersects P along hh', called the horizontal line. A vertical plane V through S_1 intersects P along the line vv', called the principal line. The intersection of the principal line vv' and the horizontal line hh' gives the principal point E, and S_1E is then the distance line according to the definition already given. In practice, to locate the principal point E on the plate of a phototheodolite we photograph a pair of cross-wires.

Fig. 3 shows how to find the position of the plate for plotting at station S_1, if the distance line f and the length of the base line S_1 to S_2 are known. The base line may be drawn to any scale, but the distance line f must be the actual distance. The point s_2 is the projection on the photographic plate of the position S_2. The distance s_2E is scaled from the photograph and the perpendicular to the base line, $RT = s_2E$.

Again referring to Fig. 2, we find that the horizontal projection of a on the plane of S_1 is a', and of A in space A'; aS_1 and AS_1 are in a straight line and the triangles S_1aa' and S_1AA' are similar for obvious reasons. We then have the relation:

$$AA' : S_1A' = aa' : S_1a' \text{ and}$$

$$AA' = \frac{S_1A' \times aa'}{S_1a'}$$

Therefore, if we have the horizontal line hh' and the point a on our plate we know aa' by direct measurement,

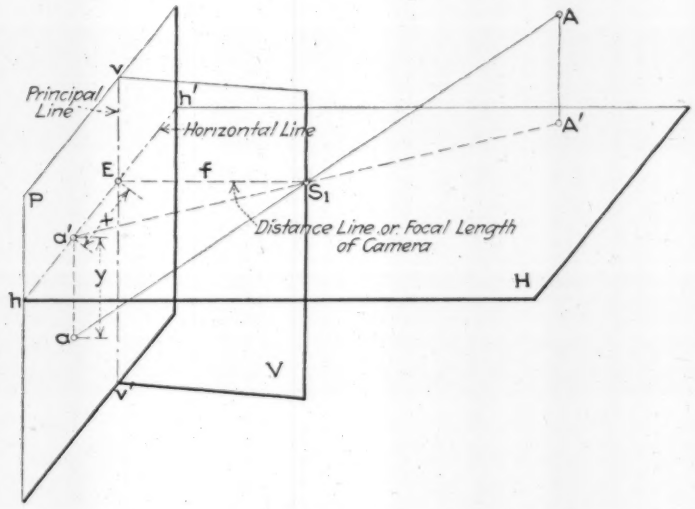


FIG. 2. PHOTOGRAPHIC PLATE IN POSITION, ILLUSTRATING ELEMENTS OF PICTURE

also the lengths S_1a' and S_1A' may be determined, knowing f and x, and we are able to calculate or determine graphically AA', which is the elevation of the point A in space above the height of instrument.

These are the bare elements of photo-surveying; we will now describe their application to a few surveys.

AVALANCHE PROTECTION WORKS ON THE ARLBERG RAILWAY

Shortly after the completion of the Arlberg railway in Northern Tyrol, considerable trouble with avalanches developed, which made the railway service unsafe. In the year 1889 the officials of the Austrian State Rys. sent Vincenz Pollack to prepare the necessary plans for constructing protection works. A part of the slope exposed to avalanches is inaccessible, being very steep, rocky and full of deep gulches. It is located on the western slope of the Arlberg and therefore open to the west winds, which are laden with rain. Hardly one day passes without rain.

This survey was made as follows: Instrument positions were selected on the opposite slope of the valley, special attention being paid to getting good views of the different gulches. The distances between the different positions were so arranged that the sights did not intersect at too small angles. The camera positions were tied into the railway center line by horizontal and vertical angles. The distances from the camera stations to the photographed points were from 1000 to 1500 m. Some mountain tops were from 2000 to 2500 m. and more away.

A sufficient number of points on the slope were located, their elevations calculated with a slide-rule and a contour map made. As a rule there were three and more sights for each point, so that checks were possible. There were only a few cases where triangles of error occurred, and these, as a rule, could be traced back to inaccurate drafting.

LOCATION OF THE JUNGFRAU RAILWAY ALONG THE EIGERWAND AND EIGER GLACIER

This work was done by the late Dr. C. Koppe, professor of geodesy at the Polytechnic Institute of Braunschweig, who also had made the triangulation for the St. Gotthard tunnel. For the work he used a phototheodolite, a combination of camera and theodolite, which enabled him to take pictures with camera inclined. He started his field work in August, 1896, and finished it in the first week of September, a little more than one month. The difficulties of a survey to locate exactly a railway which very often approaches an inaccessible rock wall, and at different times appears to daylight in form of galleries at places where an overlooked crevasse would carry the line out into space, can be easily imagined. Along with the necessary triangulation, very extensive photo-surveys were made so that it was possible to locate on a contour map 1½ miles of the railway line in an entirely inaccessible region.

Possibly quite a few of the readers of this article have taken a trip on the Jungfrau Railway and admired the grand scenery which opens from one of the gallieries known as the Eismeer station. It was a condition of the survey that the line should strike that point of the Eismeer, as only there the rock formation is solid. In places it was necessary to take the photographs from as steep an angle as from 30° to 40°, to avoid going back too far for the pictures to show the necessary detail. Some pictures were taken with telescopic lenses, which enabled the professor to study closely certain groups of rock. The photographs were developed in the field every day.

The triangulation stations were marked with 3- to 10-cm. gas pipes from 30 to 40 cm. long, with an iron cross at the bottom and were embedded in concrete. The photographs of the Eigerwand and Eiger glacier were made

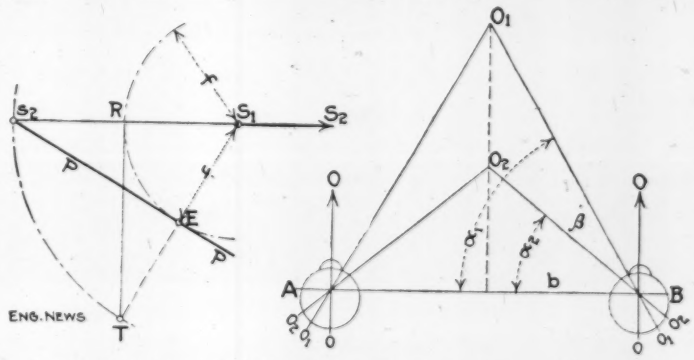


FIG. 3. PLACING PLATE IN POSITION FOR PLOTTING
FIG. 4. STEREOSCOPIC EFFECT OF DISTANT OBJECTS ON THE RETINAS OF THE EYE

from 12 such triangulation stations as well as from five other stations tied into this triangulation. One hundred good plates were the result of this work, but Prof. Koppe did not finish the whole survey, as difficulties arose with the promoter who, unable to grasp the engineering side of the enterprise, wanted the remaining work done in a less scientific way.

PHOTO-SURVEYS FOR PRELIMINARY RAILWAY WORK IN RUSSIA

In Russia photogrammetry is used very extensively. As far back as 1891 it was used for preliminary railway

work in the Caucasus. The pioneer of photogrammetry in Russia was the late Counsellor of State, R. Thiele, a civil engineer, who later achieved international fame by his land and river surveys by means of kites, one of the most promising branches of photo-surveying. Thiele applied photogrammetry to preliminary surveys in Transbaikalia for a part of the Trans-Siberian railway in the year 1827. The work included a strip of land about 400 km. long from Usty-Anon to the Chinese border. The intention was to connect the Siberian railway projected along the Amur River with the proposed Chinese railway.

Two field parties were formed, each consisting of an engineer as chief-of-party, two students of the School of Communications, one helper and one mechanic, besides the necessary number of native laborers. The country offered splendid opportunities for photo-topographical work, being formed of almost continuous chains of conical hills and covered with short grass without bush or trees. The highest hill tops were not more than 200 m. above the valley bottom. The elevations of the stations were taken with Goldschmidt and Naudet aneroids, thus giving a good check. Prominent points, which could be seen from many stations, were sighted at, and vertical angles carefully read. On account of bad water the plates were not developed in the field. From July 22 to Sept. 1, 128 km., including the triangulation work, were surveyed in 17 working days from 96 stations; and the triangulation was checked by two additional base-line measurements.

On a new survey, Thiele kept only his mechanic. New laborers were engaged, the summer tent was replaced by a round felt tent called "yurte," used by the Burjates, a nomad tribe. The purpose of the new survey was to avoid a crossing of the Argims, a very wild, as well as deep stream, and the tunneling of a mountain, called Schiwi-Guntej. Thiele finished this survey of 174 km. in only one month of 18 working days, in which time he took 83 full panoramic views, and measured four base lines. The weather in general was very favorable, but for three days there were snow storms, which stopped all field work. The camp was moved every day.

The Minister of Communications wanted the plans very urgently, to enable a decision as to which line would be the cheapest to build, so Thiele did not prepare a contour map, but made a map, showing merely the watersheds and principal elevations. It took him a month and a half to prepare this plan. It was never completed, as the government abandoned that line. The map of the second survey, however, was carried to completion, and the precision of its contours was examined later by plotting the railway line as determined by a level. This line fitted to the contours in all details. Thiele, as well as his former assistant, Schtschuroff, made many other photo-surveys for railways.

Advantages and Disadvantages of Photo-surveying

We have given the foregoing descriptions in order to show the great difficulties the engineer would have to overcome in countries far away from civilization and without means of communication, if a heavy camp outfit had to be carried along. In these instances, splendid results were obtained with very small surveying parties in an incredibly short time.

While the fieldwork did not take up much time, the office work was painstaking and very tiresome. The finding of corresponding points, the determination of their coördinates, the calculation of their elevations above the horizontal line, took very long, and often resulted in mistakes. With increasing lengths of base lines the difficulty also arises in photogrammetry of identifying the same parts of the ground in both photographs, because the landscape looks entirely different from widely separated viewpoints. In using a short base, we have very similar pictures, which make the identification of the same points easy; but in this case the sights intersect with such small angles that it becomes necessary to find the same points on three pictures. To avoid this the length of the base line is chosen, when possible, in such a manner that the intersection angles are not smaller than 30°. Added to these disadvantages is the one that the necessary accuracy is often difficult to obtain.

Stereophotogrammetry

A method by which the troubles arising in photogrammetric surveys are eliminated is based on the principle of making the necessary measurements on stereoscopic pictures, and is called stereophotogrammetry. The method is a modification of the principle of the Zeiss

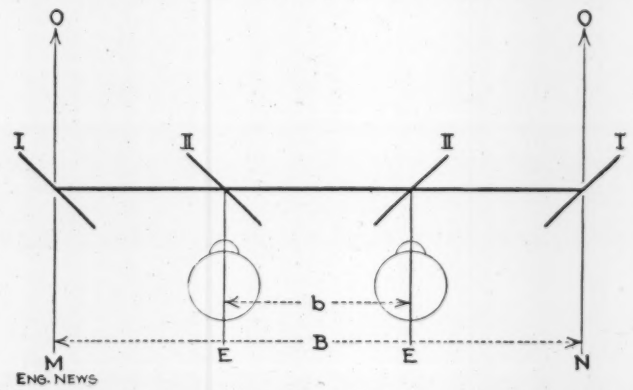


Fig. 5. Principle of the Telestereoscope of Helmholz

telemeter, which is based upon measuring distances by the difference in depth of the stereoscopic image.

If we look with both eyes at objects in different planes we perceive them to be at different distances, but if we look at them with one eye only, they appear to be in the same plane. Rays coming from objects located at a great distance, enter both eyes parallel and produce on symmetrical places of the retinas images o (Fig. 4). An object O_1 will produce an image o_1, and an object O_2 an image o_2 on different places of the retinas. This difference of position of the images on the retinas produces the effect by which we perceive relative distances. Between the position of these images and the distance of the objects, or the angles α, exists a simple relation.

The difference of the two angles α, $\alpha_1 - \alpha_2 = \beta$, is called the parallax. We are able to distinguish differences in the distance of objects to about $\beta = 30''$. It varies with different individuals, as it depends on the eyesight and on the distance between the eyes. The limits are between 400 and 500 m. We can, however, increase this ability by using glasses. If we look through a field-glass, which for instance enlarges eight times, the angle β will increase accordingly, and we will see the landscape stereoscopically for a distance of 3200 to 4000 m.

An instrument which relatively enlarges the distance between the eyes is based on the telestereoscope of

Helmholz. It consists of four mirrors, arranged as shown in Fig. 5. The observer, looking with his eyes E through II II, sees the landscape reflected from II as if his eyes were at M and N, and the eye-distance is, in effect, enlarged from b to B. If for instance $b = 60$ mm. and $B = 600$ mm., then the parallactic difference will be increased 10 times, and we will note differences in depth to a distance of from 4000 to 5000 m.

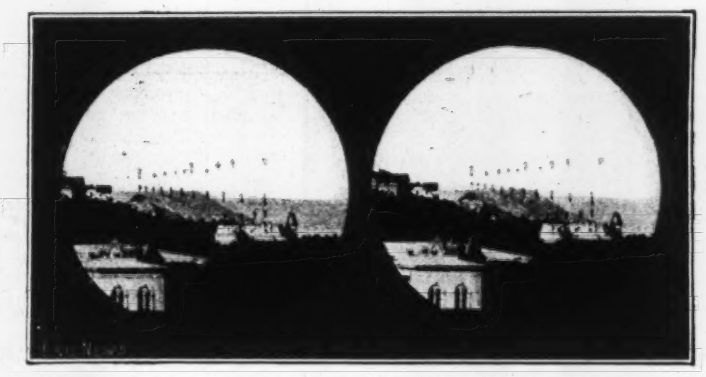


FIG. 6. LANDSCAPE VIEWED THROUGH A ZEISS TELEMETER

The Zeiss relief telescope comprises a double telescope and a telestereoscope, where the mirrors are replaced by prisms. The enlargement e and the base relation $\frac{B}{b}$ appears as a product, called the total plastic, $e \times \frac{B}{b}$, which indicates how many times the stereoscopic effect is increased. For instance, for an enlargement of eight times and the base relation of $\frac{B}{b} = 10$, we have $e \times \frac{B}{b} = 8 \times 10 = 80$. We will then see a plastic image of the landscape to a distance of about 32 km.

The converging angles increase according to the total plastic, so that it is possible to notice very small changes of the parallactic angle and to judge from this difference the difference in distance of two objects. While we are able to notice parallactic differences of angle of about 30″ with the naked eye, with a relief telescope with a total plastic of 80, we can distinguish a parallax of about 3/8″.

The principle of the Zeiss telemeter consists of measuring the parallax by stereoscopic observations of depths. The instrument consists of a relief telescope, as described, the outer prisms of which are adjusted in different models to 51, 87 and 144 cm., which form the bases for the measurement of distances. In the image plane of the eyepiece are glass plates, each a little detached from its neighbor, with numbered marks so arranged that when one looks through the eye-pieces the graduated plates form a stereoscopic picture (Fig. 6). The landscape is seen in enlarged plastic, the scale extending back into space. Distances can now be measured by direct comparison of the prominent points of the image with the scale. We do not measure the parallax, but compare the stereoscopic depth, which depends upon it, with a known scale. As we are able to notice very small changes of the parallax, it is possible to get very exact results with a short base.

This principle has now been applied to photogrammetry. We replace the images of the landscape by photographic plates, which are taken from the ends of a short base line. These pictures can be taken with the same camera one after the other. Looking at the plates through an apparatus, similar to a Zeiss telemeter, they replace the natural landscape, and the distance to any point can be measured by means of a depth scale.

THE STEREOCOMPARATOR

The instrument used for measuring distances from stereophotographs is known as a "stereocomparator," and

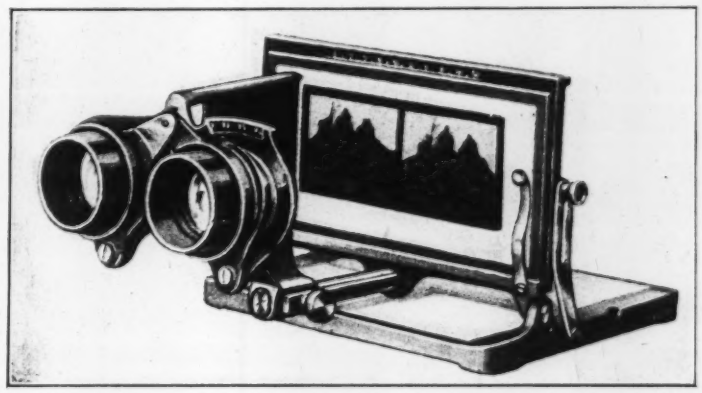


FIG. 7. AN ORDINARY STEREOSCOPE

is illustrated in Fig. 10. To understand better the working of this apparatus, Dr. C. Pulfrich, the inventor, has equipped an ordinary picture stereoscope with an attachment called the stereomicrometer, which enables a study of the principles of stereophotogrammetry and the handling of the stereocomparator. Fig. 7 shows the stereoscope without the stereomicrometer, and Fig. 8 the stereomicrometer.

The apparatus consists of a metal-frame (R) with two rectangular openings, which hold the stereoscopic photographs. The so called "wandering mark" consists of two metal pointers, which are curved in toward the pictures, so that their points come very close to the photographs. As seen through the stereoscope these pointers appear as one suspended in space.

This pointer can be brought into coincidence with any

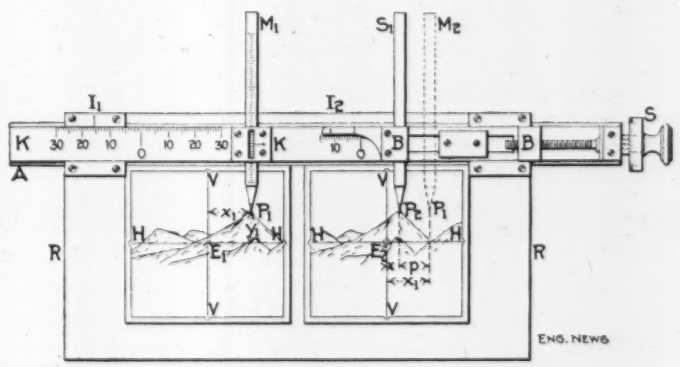


FIG. 8. STEREOMICROMETER ATTACHMENT FOR AN ORDINARY STEREOSCOPE

point P of the picture, the coördinates of which x, y and a (parallax), can be measured. For this purpose the pointers M_1 M_2 can be moved on the sliding scales I_1 and I_2. A slight horizontal movement of the right-hand pointer M_2 is possible by means of the micrometer screw S. The scales I_1 and K give the coördinates x and y of the point P; and the readings on scale I_2 of M_2 and drum S give a function of the angular parallax a. We are thus able to make stereophotogrammetric measurements on any stereoscopic pictures, but these measurements are not

exact, the use of this instrument being only to make beginners acquainted with the nature of stereophotogrammetric methods and especially with the working of the "wandering mark."

To find the coördinates of a point P, from its projections P_1 and P_2, respectively, on the stereophotographs, we proceed as follows: First set the pointers M_1 and M_2 on the principal points E_1 and E_2 (Fig. 8) and read the three scales. This is the zero position of the pointers. All other readings are thereafter referred to the zero position. In this position the pointer M in space appears to be at an infinite distance (Fig. 9), as the rays M_1 through E_1 and M_2 through E_2 are parallel.

We now look through the left-hand eye-piece and set the pointer M_1 so that it is immediately above the point P_1 of the photograph (Fig. 8). Then we look through the right-hand eye-piece and set the pointer M_2 in a corresponding position, above the picture point P_2. Now looking through the stereoscope with both eyes we see the image of the pointer at M' or M'' (Fig. 9). By moving the pointer M_2, forward or backward, by means of the micrometer screw S (Fig. 8), the image of the pointer will appear to move to or from the observer, and we can now set the pointer with great accuracy and take the different readings.

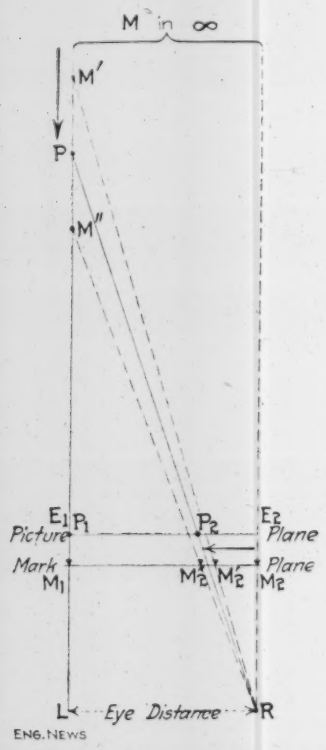


FIG. 9. PRINCIPLE OF MEASUREMENT BY THE STEREOMICROMETER

The pointer serves the purpose of a stadia rodman in the field as one can send it over the whole picture and take the measurements to any of its points. It has the great advantage over a stadia rodman that there are no physical obstacles to overcome. Furthermore, it cannot get lost, as sometimes happens with a rodman, or get outside of the control of the observer, as all its movements are made by him.

The essential difference between the stereocomparator and the stereoscope with its stereomicrometer attachment is that the pointers of the stereomicrometer move, and the pictures remain fixed, while in the stereocomparator the pointers are fixed, and the plates (negatives) and the eye-piece microscope move; but the principle is the same in both, for we are under the impression that the marker of the stereocomparator moves and not the pictures.

Figs. 10 and 11 show a stereocomparator. Both photographic plates are placed on a movable frame; I, II, I_2, II_2 are the four prisms of the mirror stereoscope K_1K_2; the images are seen through a binocular microscope, with an enlargement amounting to three or six times. By introducing a system of Porro's prisms, we see the images erect. In the image plane of the microscopes are glass plates e, marked with a vertical line, which appears through the eye-piece as a stereoscopic image suspended in space, and takes the place of the pointers of the stereomicrometer, already explained.

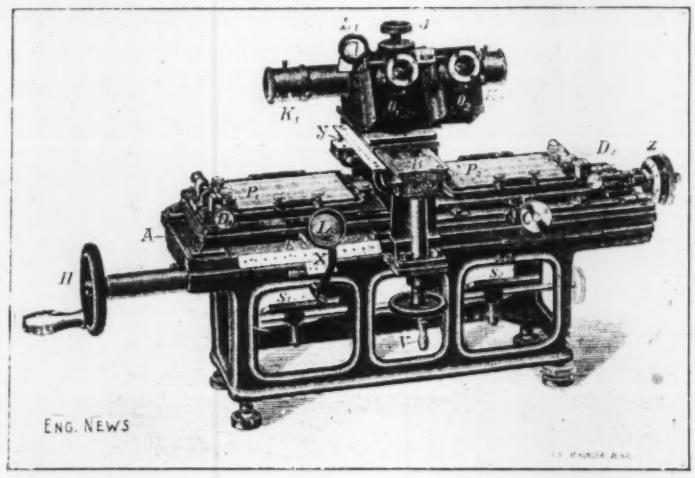


FIG. 10. A STEREOCOMPARATOR FOR MEASURING DISTANCES AND ELEVATION OF STEREOPHOTOGRAPHS

A Table carrying the two plates P_1 and P_2, sliding horizontally in the direction of the length of the plates; operated by the handwheel H.
B Table of the Stereocomparator carrying the stereomicroscope and sliding horizontally in a direction at right angles to A; operated by the handwheel V.
C Setscrew compensating differences in height on the two plates, in consequence of the unequal altitudes at which the stereophotographs were taken.
D_1 and D_2 Setscrews adjusting the plates P_1 and P_2.
E Clamp fixing the slide which carries the plate P_1 and which is shiftable in the direction of the main table A.
S_1 and S_2 Two illuminating mirrors revolvable about a common axis.
x Scale with adjustable vernier for reading off the abscissas.
y Scale for reading off the ordinates.
z Micrometer screw with adjustable measuring drum to measure parallax.

It is a condition of the instrument that the position of the two plates in the movable frame be identical with the position of the plates in the camera when the photographs were being taken, because only in this case we have comparable pictures; that is, both plates are in the same plane, so it is necessary that the camera at the two ends of the base line should have been in the same vertical plane parallel to the base line. Photographic negatives are used because it would be impossible to get positive prints, which were not subject to inaccuracies which do not occur on the negatives.

USE OF THE STEREOCOMPARATOR

To get the necessary data for calculating the positions and elevations of the different points shown on the photographic plates we proceed as follows: The plates are placed on the movable frame (Fig. 10), so that their horizontal lines are parallel to the X axis of the instrument and that their principal points E_1, E_2, lie in the direction of the principal rays (the rays perpendicularly through them). To accomplish this the hand-wheel H is provided, which moves the both plates together horizontally; the

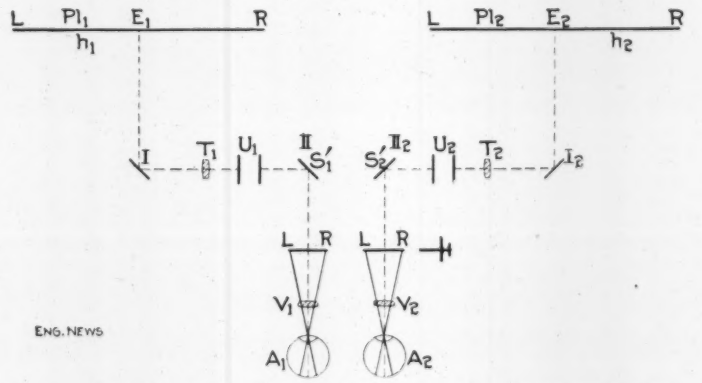


FIG. 11. DIAGRAM ILLUSTRATING THE CONSTRUCTION OF A STEREOCOMPARATOR

screw C changes the position of the right-hand plate in a horizontal direction at right angles to the horizontal movement governed by H, thus permitting the plates to be adjusted so that the same point on the two plates may be brought into the same vertical plane through the instrument, or the same horizontal plane of the landscape; by means of the hand-wheel V the stereomicroscope is moved on the Y axis of the instrument, thus bringing the principal rays to any point within the width of the plates, corresponding to a vertical movement (or change in elevation) over the landscape.

Having placed the plates as described, we read the coordinates X, Y and the parallax a on the respective scales x and y and the drum Z, which, if the instrument is in adjustment, are on zero points. Then, as with the stereomicrometer, we place the wandering marker over a point in the plastic stereoscopic picture by moving the hand-wheels H and V and the micrometer screw Z. Instead of moving the right-hand pointer as in the stereomicrometer, we move the right-hand plate, independently of the left-hand plate, by the micrometer screw Z. The readings on the scales X, Y and Z enable us now to figure the position of the point P in space.

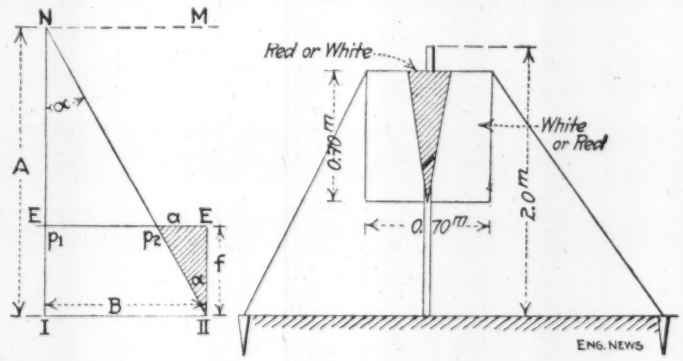


FIG. 12. DIAGRAM SHOWING PRINCIPLE OF STEREOSCOPIC MEASUREMENTS

FIG. 13. TYPE OF SIGNALS FOR STEREOPHOTOGRAPHIC SURVEY FOR BAGDAD RAILWAY

We find the folowing relation from Fig. 12: B is the base line, I and II the stations from which the photographs were taken and f the focal distance of the objective. If the plates in the stereocomparator are placed in such a way that the picture p_1 of the point P lies in the perpendicular through the center of the left-hand eyepiece (the principal ray); and p_2 is the same point on the plate at the right-hand side, then a is a function of the parallax a as measured by the comparator. If we draw two rays from I and II through p_1 and p_2, we do not get the point p, but a point N in the horizontal plane of I and at a distance A from the base. The distance of this point N from I is $A = B \cot \alpha$, and $f = a \cot \alpha$; $\cot \alpha = \frac{f}{a}$, and therefore $A = \frac{Bf}{a}$. A only depends on a, if B and f are given. Hence it follows that all points with parallax measured by the same a correspond to the same N, and that P is located on a line, drawn through N parallel to I-II. The formula $A = \frac{Bf}{a}$, therefore, gives us the perpendicular distance of the point P from the end of the base line and we find its position on the line NM from the x abscissa of the point on the picture at the left. The elevation of P above the station I is found from the ordinate y of the left-hand picture.

$$H = y\frac{A}{f} = y\frac{B}{a}$$

LENGTH AND MEASUREMENT OF BASE LINE

The necessary length of the base line is found by differentiation:

$$A = f\frac{B}{a};\quad dA = f\frac{B}{a^2}da;\quad a = \frac{fB}{A};\quad dA = \frac{A^2}{Bf}da;$$

which shows that the error in locating points increases with the square of the distance and diminishes with increased lengths of base line and focus. In this way we are able to compute the required length of base, assuming the limit of error. For instance, when $f = 240$ mm., the parallax 0.01 mm., which means $da = 0.01$, and allowing an error of 5 m., in a distance of 3000 m., we have:

$$B = \frac{A^2}{fdA}da = 75\text{m.}$$

In locating points still farther away the errors increase; for instance, it is 9 m. for 4000 m. and 14 m. for 5000 m. The length of the base line should be kept within 80% of the theoretical one.

The measurement of the base line must be fairly accurate, although no more so than required in ordinary traverse work. According to German experience the error of measurement should never exceed 1 in 1000, an accuracy which may be obtained on level ground with a steel tape. In connection with the field work of stereophoto-surveying, Dr. Pulfrich has invented a horizontal rod-measuring device, 1 m. long, which can be set up on one of the tripods of the phototheodolite over one station at right angles to base line. By stadia methods, using the phototheodolite at the opposite end of the base line, the length of the base line may be read directly on the rod to the required degree of accuracy, by means of marks moved by micrometer screws.

ADVANTAGES AND USES OF STEREOPHOTOGRAMMETRY

The advantages of stereophotogrammetry are the facilities for finding corresponding points, which with photogrammetry takes great practice and is a very painstaking work, and the possibility of locating a point on comparatively level ground with the same accuracy as a prominent point. It is possible to follow any line by locating points of same. The fieldwork is also reduced, as the number of stations can be reduced.

Before this invention photogrammetry was restricted to work in mountainous country without vegetation; it can now also be applied to wooded, hilly and mountainous country. With this method photographs need not necessarily be sharp, for exposures made with poor light and from bad positions are giving good results in the stereocomparator. Instead of a perspective picture we have before our eyes a landscape as seen in nature, only on a smaller scale, like a plastic reproduction.

The accuracy of stereophoto-surveying as found from actual experience is fully up to the standard of a good stadia survey. The location of points at a distance from the instrument of 1000 m., using a base line 50 m. long, and a phototheodolite having a focal length of 241.5 mm., was found to be uncertain within 0.8 m. In railway-location surveys with average distances of 200 to 400 m., and base lines of 25 to 50 m., the accuracy has

been found to be within 0.2 or 0.3 m. for distances and 0.07 to 0.12 m. for elevations.

Dr. Pulfrich brought the stereocomparator before the public in 1901. It is an apparatus which has proved of great value in other branches of scientific research work, especially in astronomy. Dr. Pulfrich discovered a new planet with it, by comparing stereoscopic pictures of the sky. At present stereocomparators especially adapted to star photography are found in the great observatories all over the world.

Eleven years have passed and stereophotogrammetric surveying for railways has now been introduced in Austria, Germany and Russia. In order to illustrate better the foregoing explanation of stereophotogrammetry, we will give a notable example of a stereophotogrammetric survey—that for the Bagdad railway in Asia, executed by H. Luescher.

Stereophoto-survey for the Bagdad Railway

The survey by stereophotogrammetric methods covered a part of the Taurus in Asia Minor. Although stadia surveys of this section already existed, it is a very wild country, partly inaccessible and with many deep gulches. Luescher used the following instruments for his work: One Zeiss phototheodolite, 9 cm. x 12 cm., with micrometers; three tripods; a horizontal measuring rod; two stereocomparators, 9x12 cm.; two small and one large signals; furthermore, the necessary instruments for reconnoissance, consisting of a telemeter and an iconometer, with a small planetable. The intention was to investigate how stereophotogrammetric surveying would compare with a stadia survey.

This survey was made for preparing maps to the scale of 1:500 upon which to base estimates for the necessary bridges and retaining walls. There was very little vegetation and no difficulties arose in selecting positions commanding complete views of the different gulches. The strip was about 600 m. long, the average distance to the section photographed was about 150 m.; five stations were necessary.

The survey was preceded by a careful reconnoissance, for the selection of the right instrument stations was of the greatest importance, especially as to light conditions and position of sun. Photographs taken against the sun are avoided when possible. The average distance to the part of the landscape to be photographed was found with a Zeiss stereoscopic telemeter with an enlargement amounting to eight times and a base of 50 cm. The iconometer served to find the width of a strip which could be photographed from one double station, and to select the proper distances between stations for taking pictures, with a sufficient allowance for overlapping. The iconometer used consisted of a frame, corresponding to the size of the plate, and of a diopter placed at the focal distance of the objective of the phototheodolite, so that in looking through one could see exactly what the photograph would cover.

Fig. 13 shows the type of signal used. The size of the signal depended on the distance. The ends of each double station were connected with a footpath, so as to be able to transfer the phototheodolite (Fig. 14) quickly from one end to the other. Triangulation with necessary levels served as skeleton for the survey. The stations were not necessarily triangulation points, but if not they were carefully tied to the system.

It was proven by this stereophotogrammetric survey that the field as well as the office work was considerably shorter than if it were done by stadia; besides that the resulting contour maps showed much more detail than those prepared from the stadia survey.

Other Stereophoto-surveys

Stereophotogrammetry was very extensively employed by Russian government engineers in preliminary surveys for the Amur railway in Asia. Three parties were sent into the field and many miles of territory were mapped from stereophotographs.

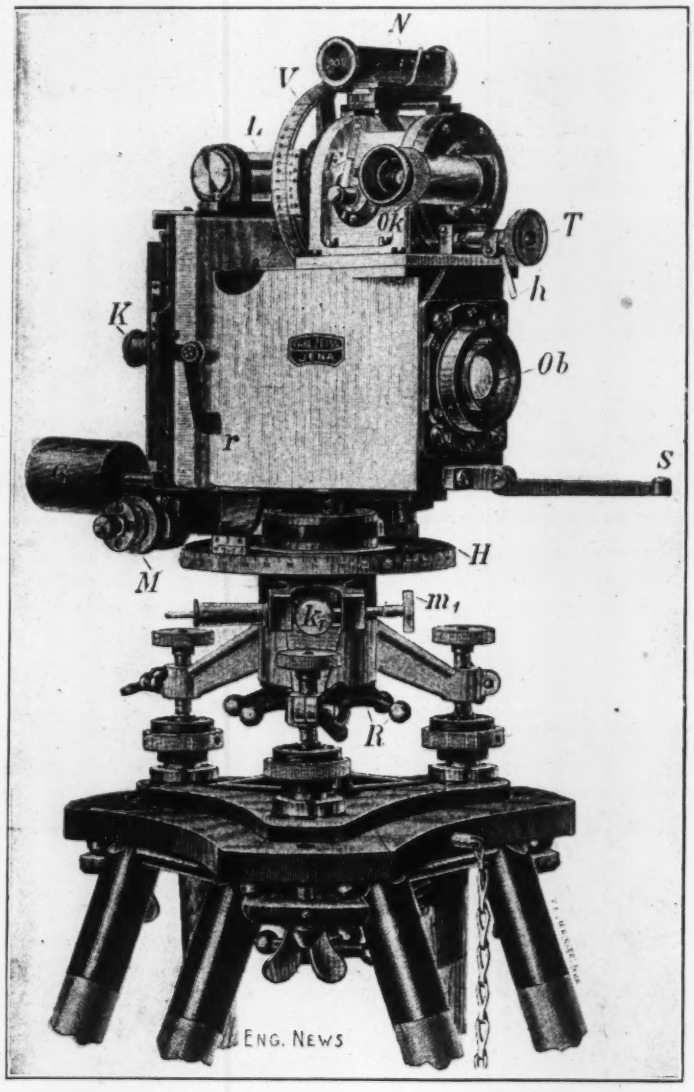


Fig. 14. Phototheodolite Used for Stereophotographic Surveys

The Austrian government was enabled to conclude the preliminary surveys for the state railway through the Dinaric Alps along the Dalmatien coast by stereophoto methods, where the ruggedness of the country had practically prevented a satistactory survey by ordinary methods.

On a survey for the preliminary location of a branch of the Austrian State Rys. in Tyrol, a locating engineer, with one assistant and four helpers, using a phototheodolite, was able to complete work in a few weeks, where stadia parties had been unable to get the requisite data in a whole season's work. On this survey it was found possible to complete about 3 km. of reconnoissance work a day. The preliminary survey as a whole was done

with a progress of 1 km. per day. The maximum distance from the instrument to the landscape photographed was 600 m. The country was very rough and the necessary grades could not be obtained without extensive development.

Many surveys for water-power development in the Alps Mountains have been made by stereophotogrammetric methods. Extensive topographical surveys have been made by stereophotographic methods in German Southwest Africa. The German, Austrian, French and Russian armies all make use of stereophotographic methods for obtaining maps from balloons, and recently the

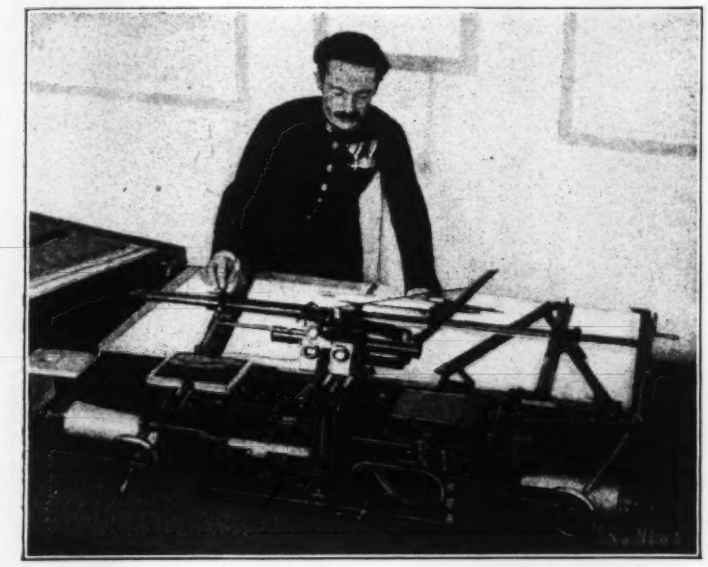


FIG. 15. THE STEREOAUTOGRAPH AND ITS INVENTOR, CAPT. RITTER VON OREL

British government has purchased a complete aëronautical stereophotographic surveying outfit for similar work.

THE STEREOAUTOGRAPH

There is another improvement which still more reduces the office work of using the stereocomparator. In working with the stereocomparator, we get the necessary data from different scales and then calculate the distances of the points from the base line by the formula $A = \frac{Bf}{a}$, before we are able to plot these points on the drawing.

Dr. Pulfrich invented some years ago a drawing apparatus which enables one to plot any point directly from the values read on the different stereocomparator scales without preceding calculations. But by a still more recent improvement the readings on the different scales are also eliminated and the plotting can now be executed automatically. As has been shown, in order to find the elements of a point we have to move the frame and set the marker. If all these movements are transferred directly to a drawing apparatus automatically, all the computation is avoided. Captain Ritter von Orel, of the Austrian Military Geographical Institute, is the inventor of an apparatus to accomplish this. He transformed the distance formula $A = \frac{Bf}{a}$ or $A : B = f : a$, which would give impracticable intersections, into $A : Bx = \frac{f}{x} : a$, in this way increasing the intersection angle in the same proportion as the base line. This modified distance formula is the basis of his instrument, which he calls a stereoautograph (Fig. 15).

The stereoautograph is connected with a drawing board, and transfers all the sliding movements through levers by an arrangement similar to a pantograph (Fig. 16). The wheel on the left (Fig. 15) moves the frame containing the two plates (negatives) and gives the X ordinate as already explained. The wheel on the right moves the plate on this side only and thus gives the stereoscopic parallax, taking the place of the micrometer drum. The lever-arms visible in the background transfer these movements to the drawing pin, on which Capt. von Orel's right hand is placed.

Fig. 16 shows a diagram of the stereoautograph. The horizontal drawing board is connected with the lower supporting part of the stereocomparator. The pin joints I, II and III are located on a line parallel to the X axis of the instrument and are the turning points for the lever-arms which are actuated by the three movements X, Y and a. The movement of the arm through I traces the horizontal projections of the points of the landscape of the left-hand plate; II their vertical projections, and the movement of the arm through III is a magnified function of the parallax.

Fig. 16 shows the instrument set on a point to the left of the principal line a distance x_1, and above the horizontal line a distance y_1. The distance f equals the focal distance previously described, and the distance A depends on (Bx) the chosen scale. The X movement of the stereocomparator is transmitted by the sliding guide F_1 to the lever turning around the axis I. The movement of F_1 measures x, the tan α (Fig. 16), while the movement of the arm reproduces the angle α directly on the board, and the angle α can be read on protractor T_1. This corresponds to a direct horizontal angle measurement between a perpendicular to the base line at the station and the point of the landscape on the left-hand plate.

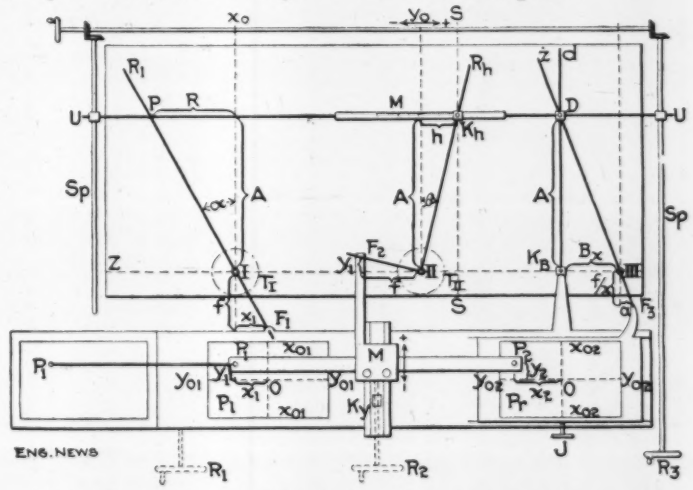
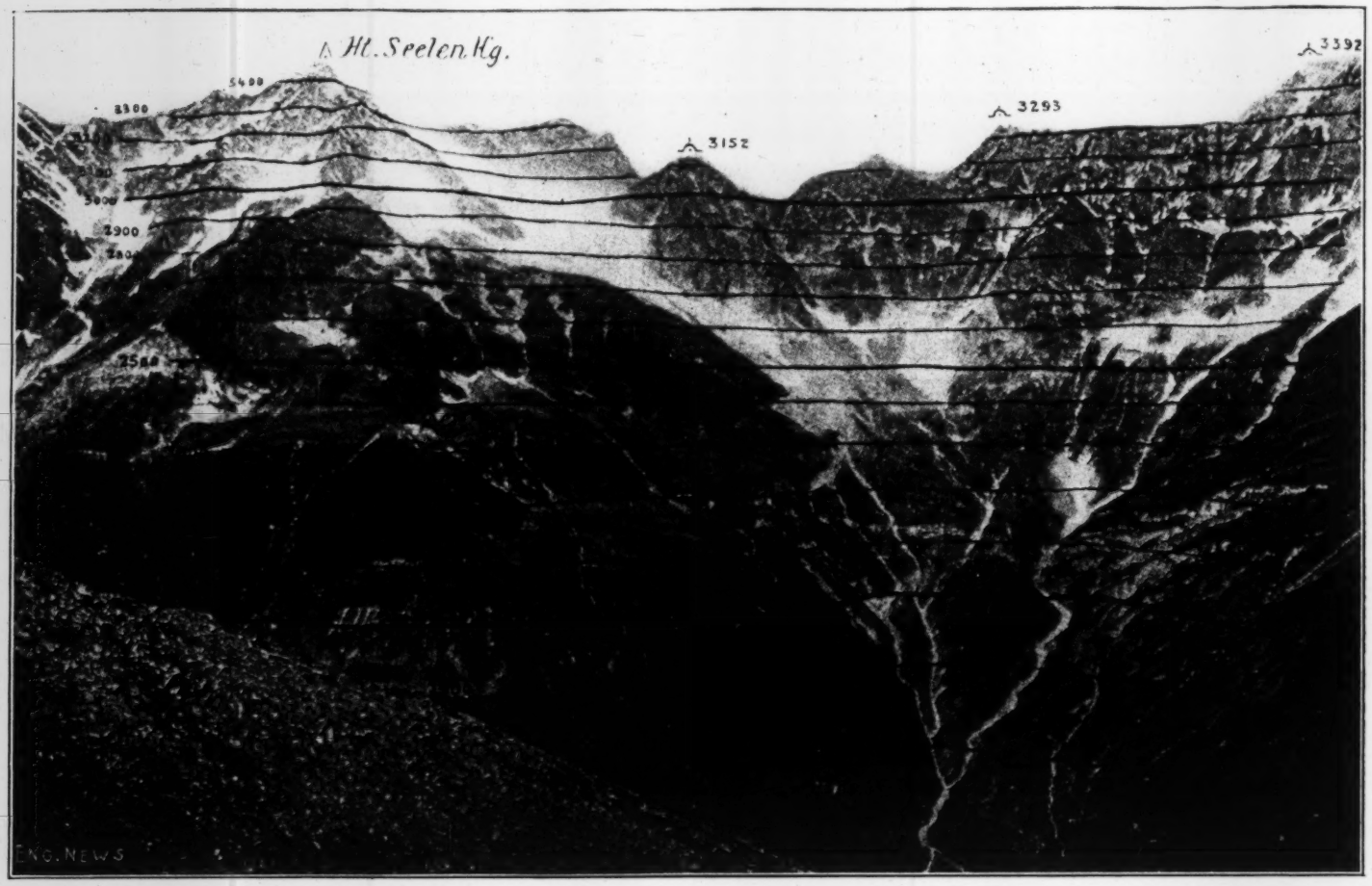


FIG. 16. DIAGRAM SHOWING CONSTRUCTION OF THE STEREOAUTOGRAPH

As soon as the marker is set, the position of the lever R_1 gives the direction of the point P. In a similar way we find the vertical angle. By turning wheel R_2 the movement of the slide Y is transmitted by means of a lever R_h, which is connected by a right-angle arm at F_2 with the Y slide; its axis also is located at the focal distance f from F_2. This movement reproduces automatically the vertical angle from the horizontal plane of the station to the point in the landscape.



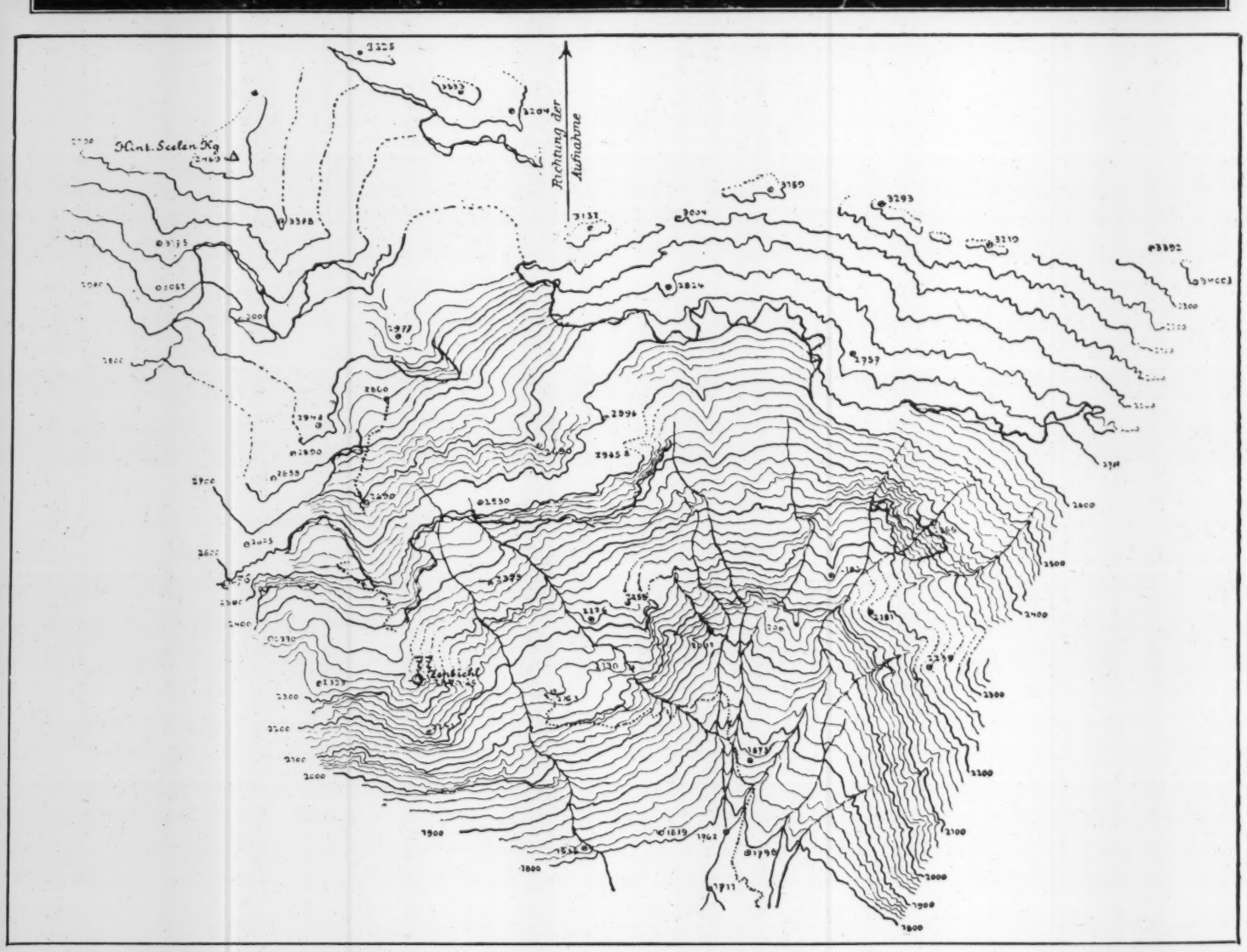


FIG. 17. EXAMPLE OF A STEREOPHOTOGRAPHIC SURVEY; ORTLER MOUNTAINS IN TYROL

The setting of the marker on any point on the left-hand negative thus gives the horizontal and vertical angles of the point automatically. The position of the point itself depends upon the distance A and is found by a system of gears which carry a long straight-edge, each end of which is attached to a worm-screw carrier. The carriers travel along the screws S_p, and are moved by means of the wheel R_3, called the depth wheel. The carriage on which the right-hand negative moves has no micrometer scale to read the parallax; but wheel R_3 governs the movement of the lever Z by means of the sliding carriage F_3, and moves the right-hand negative at the same time until the marker is placed correctly, when the stereoscopic parallax and required distance correspond. Here we are finding the parallax and distance simultaneously, in opposition to the old way, where we calculated the distance, which corresponded to a predetermined parallax.

The straight edge UU is parallel to the base Bx and therefore contains the horizontal projections of all points of the same parallax. We now have automatically the solution of the problem illustrated in Fig. 12; while the intersection of the line of the arm R_1 with the line NM (Fig. 12) gives the position of P. At P a pencil is provided for marking the point. On the scale M is read the relative elevation of the point P in space, referred to the H.I. To read the real elevation of P, it is only necessary to adjust the scale M in such a way that it will read the correct elevation when the instrument is set on the known datum plane. It is clear that the scales of T, h and A are always the same, as all three are dependent on the constant f and the base Bx. This latter can be changed to allow a change in the scale of the drawing. The scale is limited only by the size of the drawing board and the whole or part of the photographs to be plotted.

Contour lines may be drawn automatically. To do this the elevation scale M has a division which corresponds to the scale of the drawing. By means of the clamp K_h on the lever P_h, h can be set to any desired elevation; then by releasing the movement of the microscope carriage by unfastening clamp K_y and moving the straight-edge UU by the wheel R_3, points of the same height may be easily and quickly located. The point K_h will follow mechanically the line SS, which is nothing else than the horizontal projection of a contour plane. Shifting of the microscope carriage apparently makes the marker trace the course of the contour as seen in perspective from the station on the left.

It is also a simple matter to draw contour lines directly on a photographic print of the plate on the left. To do this it is only necessary to transmit to the print movements corresponding to the apparent movement of the marker (on the left-hand plate). For this purpose a photograph is fastened on a little board, which is connected with the X movement of the carriage, so that it follows all its movements. The arm P_1P_1 ending in a point transmits the changing y of the microscope carriage and traces the apparent vertical movement of the marker.

To find now the intersection of the contour plane with the stereoscopic picture we move R_1 with the left hand and R_3 with the right hand, starting from the left side of the picture. The wheel R_2 is not used. Wherever the marker is made to touch the plastic picture are points of equal altitude. The pencil at P will draw the horizontal projection of this cut on the plan and the point P_1 on the print the perspective course of it. Without moving the eyes from the eye-piece the two pencils will draw automatically the contour line on the map and its perspective picture on the photograph (Fig. 17).

The exactness of these contour lines depends entirely on the ability of the operator. In the first place he must be able to see stereoscopically, so that he is able to move the wandering marker with exactness along the stereoscopic picture. Every contour line can, of course, be checked by going over it a second or third time.

A new stereoautograph is in the course of construction, in which the wheel R_2 can be moved by the foot, so it will be possible to execute all three movements simultaneously.

Applications of the Stereoautograph

In 1907, the stereoautograph was used to plot the stereophoto-survey of the Ortler Mountains in Tyrol

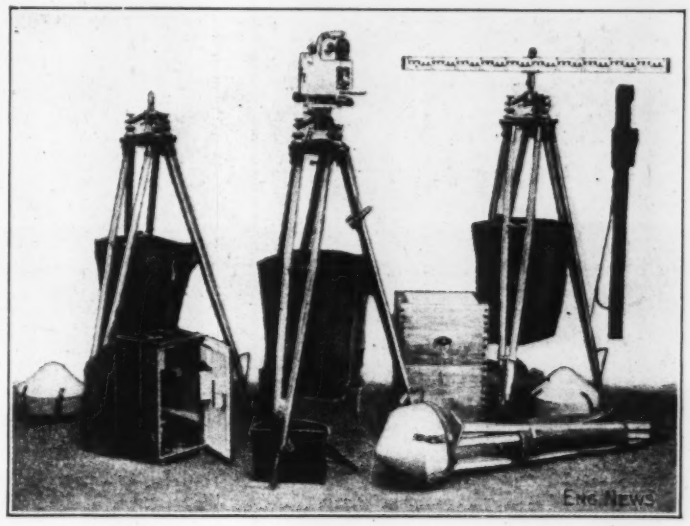


Fig. 18. Complete Set of Instruments for a Stereophotographic Survey

(see Fig. 17). The field work occupied eight days and the office work ten days. The result was a splendid map of the Ortler group, for which the International Exposition of Photography, held in Dresden, in 1909, awarded the honor prize, and it received the grand prize at the International Exposition of Means of Communications, held in Buenos Aires, Argentina, in 1910. The work was checked in 1909 by other surveying parties.

Extensive stereophoto-surveys have been made in the Austrian Alps; surveys which gave very good results when checked by other methods. In two days of field work an area 20 km. square, situated in the higher part of the "Zwickauer-hütte," in the valley of Passeier, were photographed from two double stations. The office work was completed in 16 hours. The result was a map to a scale of 1:25000, with 20 m. contours in the lower levels and 100 m. in the higher levels.

The field for stereophoto-surveys in combination with the stereocomparator and autostereograph is a vast one; to mention only a few cases, there are coast surveys from a ship with a camera placed at each end; surveys for studying the movements of glaciers, and stereophoto-surveying from balloons, aëroplanes and by means of kites. Last but not least, photogrammetry and stereophotogrammetry may be used extensively by architects, by taking interior and exterior pictures of ancient build-

ings, to enable them to make plans, elevations and sections for their own use and for posterity.

The German government created a special department in 1882, called "Königlich Preussische Messbildanstalt," which now has a very numerous collection of photogrammetric surveys of all the public buildings, churches, monuments, etc. A few years ago, this institute was commissioned by the Grecian government to survey by photogrammetry all the antique ruins of Greece, a work which was finished successfully . Photogrammetric instruments are of great value to explorers, and there is now no German or Austrian expedition sent out without the necessary instruments for photo- or stereophoto-surveying.

[Since this article was prepared for publication, we have been informed that the instruments described are to be handled in this country by the Bausch & Lomb Optical Co., Rochester, N. Y. The list price of the phototheodolite is $684, or $1137 with accessories and extras. The list price of the stereocomparator is $815.—Ed.]

A Small Water Works Plant for Kingsburg, Calif.

By C. F. Braun*

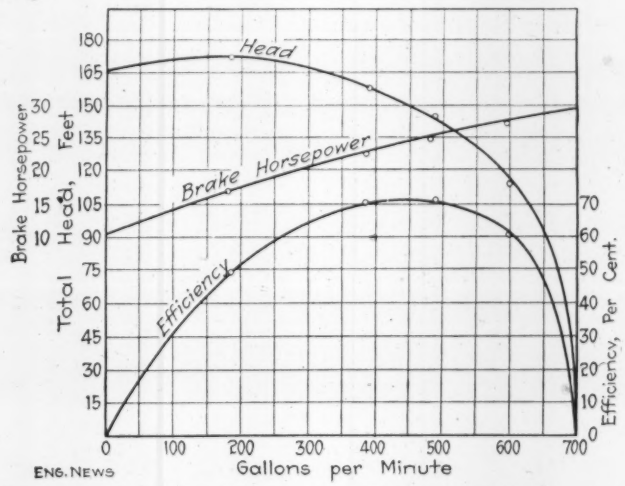
The town of Kingsburg is one of the many growing small town, in the San Joaquin valley, and has a population of about 1000. In order to keep pace with its progressive neighbors, and to secure its share of California's increasing population, the city fathers planned a system of water-works for which the people voted a $26,000 bond issue. One of the features of the plant is the pump installation. This includes two centrifugal pumps, of which one is driven by an electric motor with an automatic control, while the other pump is driven by a gasoline engine.

The supply of water is obtained from two wells about 80 ft. deep. A combined flow of about 900 gal. per minute is produced, the water being clear, palatable and only slightly hard. The surface water-level is about 8 ft. with respect to the ground and is lowered 20 ft. when the full quantity of water is being pumped. Particular care was taken that the part of the well-casing near the surface should be made tight, so that surface water should be excluded from the well, and possible contamination avoided.

Owing to the small size of the plant, automatic operation was as far as possible desirable and the electrically driven centrifugal pump was selected as meeting the requirements. The general-service pumping unit consists of an Alberger 4-in., two-stage, turbine-type, centrifugal pump, direct connected to and mounted on a common cast-iron base with a General Electric induction motor, operating at a synchronous speed of 1800 r.p.m. The motor is of the squirrel-cage type, rated at 30 hp., with an overload capacity of 25% for two hours, and operating on 440-volt, 60 cycle, 3-phase current.

The normal rating of the unit is a delivery of 450 gal. per minute against a total head of 150 ft., with an over-all efficiency from electric line to water, of 60%. Under test, an actual over-all efficiency of 62% was obtained, which is a result exceedingly gratifying with so small a unit. The pumps may discharge to the tank or mains as desired.

The characteristics of the pump are such that at no point over the entire range of discharge, even with closed discharge valve, can the pressure developed by the pump exceed the normal by more than 12%. The pump is absolutely automatic in this action, and a dangerous pressure cannot possibly be developed on the mains or pump casing.



Characteristic Curves for Alberger Two-stage Turbine Pump
(Constant speed, 1720 r.p.m.)

The head curve is unusually flat (see accompanying diagram). Variations in capacity above and below normal do not produce great variations in the pressure. This characteristic is very desirable in water-works service, for if water is pumped into the mains the quantity required is continually varying, while if pumped into a tank the head varies. The power curve shows that under no conditions of load can the power required be greater than 20% above normal, the motor and other electrical equipment thus being protected from overloads beyond their capacity. This feature is important, as with other types of apparatus the rupture of a pipe main and consequent sudden reduction of head on the pumping unit often causes overloading the motor to such an extent as to destroy it. This unit is thus made practically fool-proof without the aid of relief valves, by-passes or other protecting or reguating devices.

Such characteristics as have been described are also adapted to fire-fighting purposes. One good fire stream can be delivered at 74 lb. pressure, two good streams at 62 lb. pressure, or three fair streams at 32 lb. pressure, all automatically.

The attention required and the cost of upkeep for such a unit are practically nil. The only parts requiring lubrication are the two bearings on the pump and two on the motor. These are ring-oiling, and are constructed with renewable bronze bushings, so that in case of wear a replacement is a matter of only a few minutes. The pump is provided with a marine-type thrust-bearing to take care of any slight, unbalanced thrust that might develop, due to wear in the pump, and slight inaccuracies in manufacture. The thrust-bearing is water-cooled, which prevents any possibility of heating. The water passages of the pump, including impeller and diffusion vanes, are of bronze, carefully machined and polished, which insures long life and sustained efficiency.

*President, C. F. Braun & Co., Mechanical Engineers and Contractors, Pacific Sales Managers, Alberger Pump & Condenser Co., 503 Market St., San Francisco, Calif.

While the efficiency of such an outfit may not be quite as high as that of a plunger pumping-unit under test, the efficiency of the centrifugal pump will be maintained in actual service, whereas leaky valves, poorly packed stuffing boxes, and general wear will soon reduce the initial efficiency of the triplex to a much lower figure.

Electrical energy to operate the motor is purchased from the public service corporation whose lines serve the town. Transformers to step down the primary voltage from 4000 to 220 are placed outside the pump house.

To insure continuous service to meet the requirements of the fire underwriters calling for two sources of power, and to be independent of electric power corporations, an auxiliary gasoline engine pumping-unit was provided. The pump-end of this unit is an exact duplicate, both mechanically and hydraulically, of the motor-driven pump, so that the parts are interchangeable. The pump is mounted on a common cast-iron base, and gear-driven by a 30-hp., three-cylinder, vertical, gasoline engine, operating at 350 r.p.m. A gasoline supply-tank is placed underground outside the building.

The pumping machinery is situated in a concrete waterproof pit, 12 ft. long, 14 ft. wide and 8 ft. deep. This places the pump at the water level, so that the suction lift is reduced to a minimum, which is a desirable condition for the best operation of centrifugal pumps. A 6-in. suction pipe runs from the bottom of the pit horizontally to each well and extends 30 ft. down, terminating with a foot valve. Each suction pipe is provided with a gate valve inside of the pump pit, so that each well may be operated independently. A 3-ft. manhole and pit over each well provides access to the suction pipes and foot valves for cleaning or repairs.

A 40x40-ft. house covers the pump pit and also contains a meeting room for the city officers and a room for a hose wagon and small fire-fighting apparatus, a very convenient and economical arrangement. The house is constructed of metal lath and concrete, so that it is fireproof and at the same time inexpensive.

A 60,000-gal. steel tank on a 100-ft. tower provides an adequate reserve supply for domestic or fire service. Four converging legs with wind bracing and struts, resting upon concrete blocks, form the tower. These support the tank, which is 19 ft. in diameter, 22 ft. deep and has a hemispherical bottom of 10½ ft. radius. A conical roof gives the structure a finished appearance and excludes birds and dirt.

A float in the tank makes an electrical connection when the water level is reduced to 4 ft. below normal, and the motor-driven pumping-unit is automatically started by a solenoid-resistance-type starter. When the water level again reaches normal, the float breaks the contact and the pumping is stopped. A regular operator can therefore be dispensed with.

The city employs a man who makes service connections, collects money and attends to all general water department business, and merely looks over the pumping machinery occasionally to see that the oil reservoirs are full.

The population to be served at the start did not warrant a very elaborate distributing system. Care was taken in laying out the system, however, so that lines would be sufficiently large for fire-fighting purposes, and that extensions could readily be made. The money available for this work limited the selection of pipe to steel casing with lead joints. This was carefully dipped in asphalt so that corrosion would be retarded, and reasonable life obtained. There was laid 2000 ft. of 8-in., 18,000 ft. of 4-in., and 11,000 ft. of 2-in. pipe.

The cost of the entire plant was about $24,000. Olmstead & Gillelen, of Los Angeles, were the engineers, and C. F. Braun & Co., of San Francisco, were the constructors of the entire work.

The Milwaukee Ave. Viaduct, Chicago

The viaduct carrying Milwaukee Ave., Chicago, across a group of railway tracks has been rebuilt recently and is the first of the kind in that city in which reinforced-concrete spans have been used. The following description is from a paper read by L. K. Sherman (one of the contractors) at the annual meeting of the Illinois Society of Engineers and Surveyors:

The viaduct crosses the main tracks of the P., C., C. & St. L. Ry. and the main and yard tracks of the C., M. & St. P. Ry. and the C. & N. W. Ry. at an angle of 45°. There are ten spans, as follows, starting from the south: 66-ft. steel through girder span, 157-ft. steel through truss span, eight reinforced-concrete girder spans varying from 52 ft. 6 in. to 58 ft. The north reinforced span consists of encased steel girders. The steel spans were used over the main tracks, where continuous operation of trains precluded the use of the falsework necessary for concrete construction. The foundations for the piers rest on 50-ft. piles, which had to be arranged so that the pier footings straddle the Illinois Tunnel Co. tunnel, which runs under Milwaukee Ave. The piers are a feature of the structure. On account of car clearance on the adjacent tracks they are only 15 in. thick. Each bent consists of four of these concrete piers or columns, 15x48 in. and 15x72 in., with a concrete cross girder.

The roadway is 40 ft. wide, with two tracks for electric cars. Outside of the main girders on each side are 8-ft. cantilever sidewalks, making a total width of 61 ft. The concrete floor is about 10 in. thick, supported by reinforced-concrete girders 25 in. wide and 2 ft. 9 in. deep, placed at right angles to the piers, giving a 40-ft. span. The main girders, along the curb line, are 10 ft. 6 in. high and about 2 ft. thick, and owing to the skew these are required to support the floor girders. The main girders are supported on the piers by heavy structural steel brackets encased in the concrete. The clearance from top of rail to bottom of girder is about 17 ft.

Some complications in construction were involved. Most of the yard tracks had to be left clear each evening for switching purposes at night. In order to give the necessary clearance for cars and the team roadways, the floor and girder forms had (in most cases) to be suspended by rods from overhead falsework. Timbers, 8x16 in., were required for falsework on account of the 12-ft. spans over tracks and roadways. The foundation concrete was placed from a mixer set up on a flat car. The superstructure was built by hoisting the concrete in an elevator from a mixer near the supply track and distributed by push carts of 6 cu.ft. capacity. Electric power was used to operate the two mixers, hoist, sawmill and boring machines. There are about 4000 cu.yd. of concrete in the structure and over 300 tons of reinforcing steel, or about 125 lb. of steel per the cu.yd. The concrete in general was 1:2:4 (using crushed limestone), except 1:4 for columns and 1:3:6 for some footings. Tests of the concrete have shown over 4100 lb. per sq.in. compression, with an average of about 3000 lb. The columns are heavily reinforced with square spiral and vertical rods, four to six sets in a column, and bound with horizontal stirrups. The columns were difficult to pour and spade and required much experimenting for satisfactry results. The cement gun was used for pointing to some extent.

The structure was designed by the engineers of the C., M. & St. P. Ry., and the erection of the steel spans was performed by them. All of the concrete structure was built by L. K. Sherman & Co.

An Ordinance for the Removal of Poles and wires from residence streets of the city of Madison, Wis., has been held void by the State Railroad Commission. The commission is of the opinion that the ordinance was prompted by esthetic considerations only and, therefore, is unreasonable.

The Latah Creek Bridge, Spokane, Wash.

By J. F. Greene*

A concrete bridge of pleasing design and monumental size is now under construction across Latah Creek, in Spokane, Wash. As shown in the plan in Fig. 1, the new bridge is so located that its center line parallels an adjacent old timber trestle which is in such precarious condition as to demand replacement. The new line is curved at one end so as to connect two different streets on opposite sides of the valley.

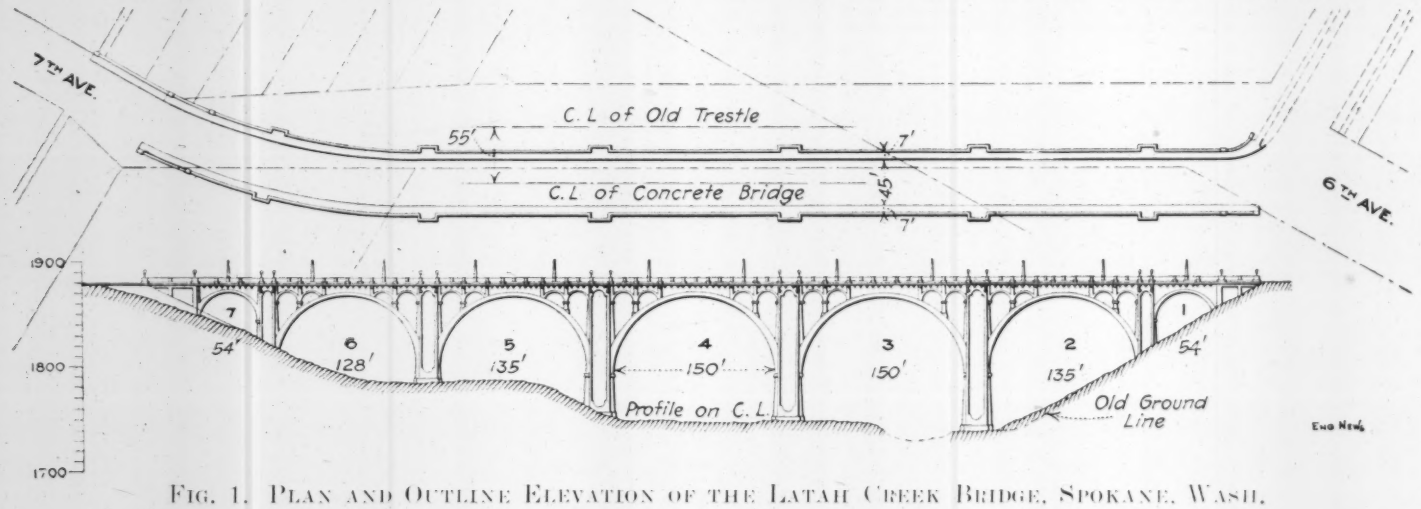


Fig. 1. Plan and Outline Elevation of the Latah Creek Bridge, Spokane, Wash.

The Latah Creek Bridge is 940 ft. long and consists of seven arch spans, varying in length from 54 to 150 ft. clear span. Two of the arches at the west end are on the curve; with their piers on radii. It has a 45-ft. roadway, with two 7-ft. sidewalks and carries a double track for heavy interurban railway cars. Each arch consists of four arch ribs, carrying the roadway slab on spandrel columns and arches.

In the determination of span lengths for the arches, architectural considerations were given much weight and an effort was made to effect a combination which was adapted to the ground line and site. The intrados and extrados curves are many centered, and were so laid out as to give a neutral axis which conformed very closely with the thrust line under dead load. The thickness of the arches are such that under any system of loads the line of resistance lies within the middle third, and the maximum stress is below 500 lb. per sq.in. Steel rods are added for temperature stresses.

Foundations—Before beginning the design, test pits were sunk which disclosed a bed of hardpan below the creek bottom and a very compact sand at other points. As a result of loading tests upon the sand, the maximum pressure of 6 tons per sq.ft. was determined upon and heavy reinforced-concrete spread foundations were designed for this condition. In Pier 1, located on the east bank, in addition to the base shown on the plan, a cut-off wall 10 ft. deep, 6 ft. wide at the top and 3 ft. wide on the bottom was built along the west edge of the pier. During excavation for this pier, the sand along the east face stood without sheeting for a depth of 30 ft.

Piers—The piers are of the box type, with solid longitudinal sections in line with the arch ribs and connecting curtain walls. Below the level of the sidewalks they are solid.

Arches—The arch ribs are spaced 16 ft. 6 in. c. to c. and in all the spans are 5 ft. wide for the outer two ribs and 6 ft. wide for the inner two. They vary in thickness with the length of the span. Each outer rib is connected to the adjacent inner one by a thin slab at the intrados, but the inner two ribs are not connected except by the floor slab. As shown in Figs. 2-3, this floor slab is of reinforced concrete, carried on beams between the arch ribs and the spandrel arches and dropped down below level to allow space for the railway track. A waterproofing mat consisting of three alternate layers of burlap and asphalt cover this slab; a waterproofing cover of 2½ in. of concrete and a 1-in. sand cushion for 4 in. creosoted-wood blocks rest on top.

The two arches on the curve are of interesting design. In them the piers are on radial lines and the four arch ribs are straight, parallel and of varying length, as shown

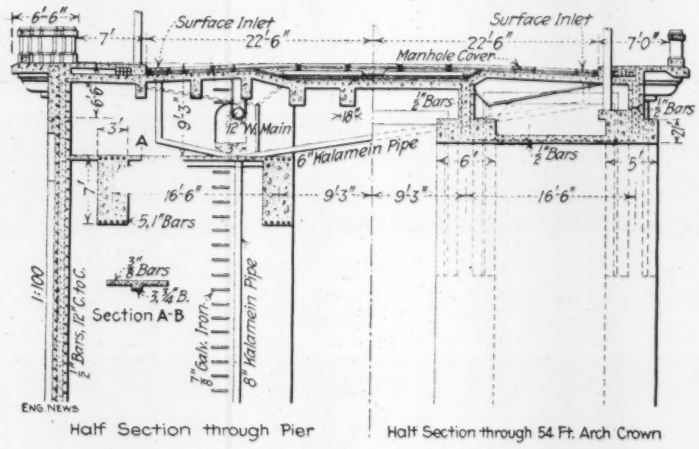


Fig. 2. Cross-section through Floor of Typical Arch

in Fig. 3. The curve is made in the transverse floor-beams spanning across the longitudinal beams, which are in line with and above the arch ribs.

Expansion Joints—In the spandrel walls, expansion joints are provided at the ends of each panel and concealed behind pilasters. The design of spandrel arches is one about which there is considerable difference of opinion among engineers. In the Connecticut Ave.

*Construction Engineer, City Engineer's Department, Spokane, Wash.

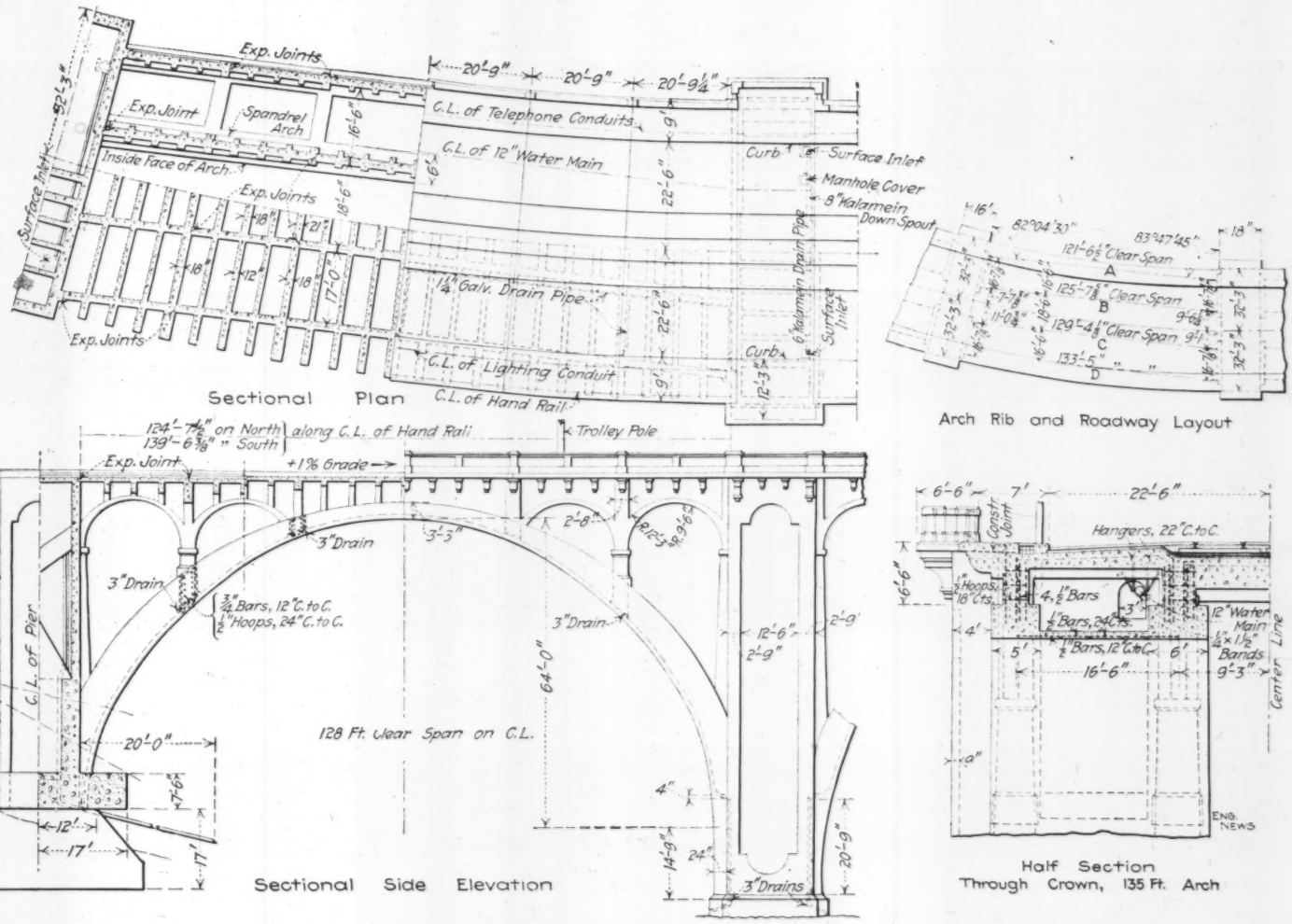


FIG. 3. DETAILS OF ARCH ON CURVE

Bridge, at Washington, D. C.,* and the Monroe St. Bridge at Spokane,† the simple treatment of the longitudinal faces of spandrel walls eliminated pilasters for concealing expansion joints. A fall in temperature, which lowers the main arch, draws the spandrel arches away from the piers, provided that there are no joints between the pier and crown of the arch.

In the Connecticut Ave. Bridge joints were left at the crown of each of the spandrel arches, which are visible at a distance of several hundred feet. On the Monroe St. Bridge, joints were placed at the third points and steel enough placed in the walls from joint to joint to take up the thrust of the two outside arches. Unconcealed joints are unsightly and even concealed joints are objectionable because it is exceedingly difficult to prevent leaks from these points.

MINOR DETAILS—Surface drains are provided at each pier and sub-drains for the expansion joints lead into these main drains. Provision has been made for a 12-duct line for the Washington Water Power Co., an 8-duct line for the telephone company; for a 12-in. water main and for a 12-in. gas main. Ornamental cast-iron posts resting on a concrete handrail support lights on the piers.

CONSTRUCTION

MATERIALS—The aggregate, consisting of screened gravel, ranges in sizes from 2½ to ¼ in. with an average percentage of voids equal to 37. The sand, screened from the same pit, is well graded and with a uniform percentage of voids equal to 33. At times owing to the difficulties of hauling over wet roads or a demand in excess of the capacity of the gravel plant, a crushed rock, showing voids of 45%, was used.

To eliminate the necessity for excessive cement-storage capacity at the bridge, an inspector was sent to sample

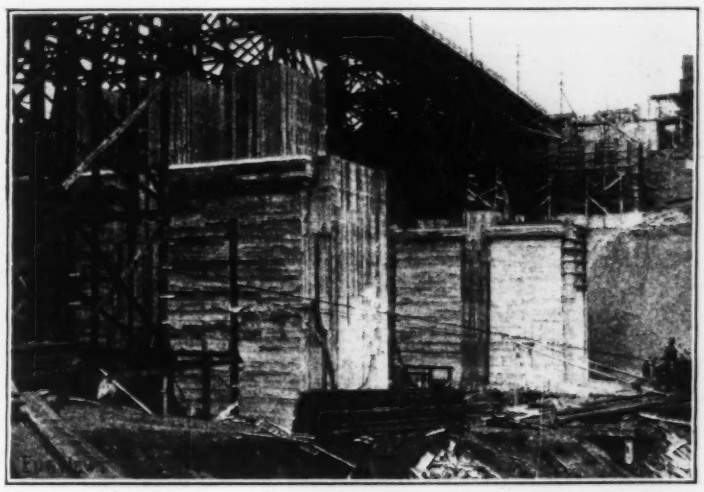


FIG. 4. VIEW OF PIERS DURING CONSTRUCTION

the cement placed in storage at the mill, the cement being shipped upon the expiration of the time required for 30-day tests; and this cement was frequently taken directly from the cars to the mixer, thereby eliminating rehandling.

*"Engineering News," June 1, 1905, p. 571.
†"Engineering News," Sept. 2, 1909, p. 241.

PLANT—The concrete plant is located at the east end of the bridge and consists of a storage bin for sand and rock, a half-yard batch mixer, a cement shed of 2000 bbl. capacity located on a railroad siding, and adjacent to the mixing plant, and an industrial railway running along on the south half of the trestle with turn-outs, and hoppers for depositing the concrete in the arches and piers.

LAYING CONCRETE—The concrete after mixing, was loaded into a train of two two-yard side-dump cars and drawn to the proper location by a gasoline locomotive. After dumping through holes cut in the deck of the old wooden trestle, the concrete flowed through open steel chutes to place.

This method of concreting produced an atmosphere of continual turmoil with the contractor owing to a natural desire to use an excessive amount of water. The water was measured in a tank with a float attachment, controlled by a 2-in. valve, and the amount was under the control of the operator. Any tendency of the concrete to stick in the mixing hopper or in the cars, or to clog in the chutes, is most readily offset by the addition of water, and a fluid mix requires little or no handling upon final deposition. During the construction of the piers, which contained about 40 yd. of concrete for each lineal foot of depth, the concrete was placed in layers of a foot at a time and an interval of an hour would elapse between the completion of pouring upon a section and the beginning of another pouring upon the same section. During this interval any excessive water in the concrete would rise to the surface and lie in pools thereon. Such a condition resulting from this or other causes is bad for several reasons. First, because this water, standing long enough to clear, deposits a layer of scum or laitance such as one may see upon a pier deposited under water; secondly, the new concrete crowds the water ahead of it, and this water in its movement washes the cement from the aggregate over which it passes. If it passes near a form, even though that portion of the concrete be spaded, streaks of wash sand will be visible upon the removal of forms. If the mixture as it is being deposited is so liquid that it will flow into place with little or no shoveling, it is very probable that there are pockets of grout, and of rock, and that there is not a uniform aggregate. Whether concrete poured with an excess of water, such as one sees on many of the jobs in which the chuting system is employed will equal in life that which just quakes under foot and no more, time alone will tell.

Under the system of inspection adopted, one man placed at the mixer, watched the measuring and mixing, checked the receiving and use of cement, and took void tests on sand and rock at two-hour intervals during the runs. From these void tests the quantity of concrete placed each day was determined and checked against the estimated amount of the run figured from the dimensions in place and a variation of more than 1% was investigated. A second inspector, located at the point of placing supervised the spading and working up of the concrete.

METHOD OF CONCRETING—The arches were concreted with voussoit sections, eight to each pair of ribs, and three keys, one at the crown and one at each third point. Each of the third point keys contained a 12x12-in. reinforced-concrete strut as a hinge. In every case, owing to the settlements of the falsework, these struts developed open cracks on the top side. The keys were composed of the same aggregate, 1:2:4, as the rest of the arch ring, but were placed very dry to reduce shrinkage.

In the process of concreting, the crown sections of the arches were placed first. As several of the arches were upon filled ground and settled 0.06 ft. under load, and as under this rib system no provision had been made to relieve the compression of the ribs owing to the settlement, there was a bulging at the third points which had to be relieved.

FORMS—The forms for the piers consisted of panels 5 to 7 ft. high, with vertical sheeting, 2x10-in. tongued-and-grooved pieces, and 3x8-in. horizontal waling spaced 4 ft. c. to c. The walings are tied by twisters of ¼-in. galvanized wire spaced 4 ft. c. to c., pulling against spreaders which are inserted between forms and removed as the concrete level rises. An objection to vertical sheeting on pier work is that the vertical board marks accentuate the horizontal joint marks between different day's runs.

The side forms for the arches were framed on the ground in panels with both the extrados and intrados cut to curve. These forms were tied together across the top with 2x8-in. planks, cleated to the lagging at the bottom and tied with galvanized-wire twisters at an intermediate level. The forms for the inner walls of the arch ribs were held up by temporary legs resting on the lagging and were removed as the concreting progressed, the weight of the forms coming upon the 2x8-in. ties which connected the outer forms. The lagging for the extrados was nailed on the backs of the side forms.

FALSEWORK—The falsework consisted of 8x8-in. posts spaced about 14 ft. centers longitudinally, and 5 ft. centers transversely. Upon these posts were placed corbels, 3 ft. long cut to curve, over which was built up a rib consisting of seven planks 1⅝ in. by 12 in. by 16 ft., nailed together and bolted with ⅝-in. bolts spaced 18 in. c. to c. It was intended that these ribs should carry the load to the posts by arch action, and provided that the ends of the ribs could have been fixed, there was sufficient section to do the work safely. During the process of adjusting the lagging it was found possible to take the arch out of these ribs with light wooden wedges, and in view of this, the contractor was required to reduce the spans of the ribs to one-third the original length by inserting 4x6-in. struts running from the bottoms of the ribs to the adjoining posts.

The erection of falsework was undertaken upon Arch 6 first, located at the west end of the bridge and upon a curve. This arch has a span of 128 ft. on center line, with its west face at springing line upon a radial line forming an angle of 82° 4′ with the center line. Both of these conditions, namely the beginning of the work and the peculiarity of the span, combined to make the unit cost of erection of falsework in this span 3.2 times that of Span 4, and the labor cost on forms 1.4 times that of Span 4. From a standpoint of economy, it seems evident that radial piers are not desirable.

CONSTRUCTION METHODS—A comprehensive cost report of the job has been kept; a daily report, embracing total quantities and cost for each individual portion of the work, and a monthly report upon which the totals for the month are drawn off, the unit pieces for the different classes figured and the percentages of the completed

portion to the whole estimated. Duplicates of all reports are made, one copy for use at the bridge, and one for the city engineer's office files.

A diary is also made out daily containing a record of accidents, discussions and disagreements, and whatever else that might prove of use or interest.

PERSONNEL

The contractor for the bridge is J. E. Cunningham, of Spokane; the contractor's superintendent, J. C. Stack. The design and construction of the bridge were under the direction of Morton Macartney, City Engineer, the design having been made by W. S. Malony, and checked by Waddell & Harrington, Consulting Engineers, of Kansas City, and the construction in charge of J. F. Greene, with W. H. Fisher as assistant. The contract price for the whole structure was $416,000.

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A 200-Mile, 90,000-Volt, Direct-current Power Transmission Project in Sweden and Denmark

The Danish government, to meet the increasing demand for electric power in Copenhagen, for some time has been negotiating with the Swedish government for the sale and transmission of electric current from the new large hydro-electric station recently built by the latter at Trollhättan. The report of the engineers, retained by these governments to study the feasibility of the project, has now been made and is abstracted in the *Electrical World* of Feb. 8, from which the following is arranged.

LINE OF THE PROPOSED TROLLHATTEN-COPENHAGEN 90,000-VOLT DIRECT-CURRENT TRANSMISSION LINE

The Trollhättan plant is now equipped with eight 10,000-hp. turbine units, with space provided for two more of the same capacity. It is proposed to transmit to Copenhagen this spare 20,000 hp. At the outset additions to the existing steam plant were considered, but it was found that the cost of generating such a local supply would be made up of a fixed annual charge of $5 per kw., plus 0.4c. per kw.-hr.

In order to make the transmission project feasible, a nearly continuous load was found necessary on account of the large expenditure in the transmission line. It was found that about a third of the local maximum demand, or 11,600 kw., could be maintained during 5200 hr. per year, or for some 14 hr. per day, giving a load factor of nearly 60% for this portion of the local service. The 11,600 kw., with transmission losses, corresponds to about 20,000 hp. at the Trollhättan plant, and this would absorb the output of two generating units not yet installed.

The route of the proposed line, shown in the accompanying sketch map, would be along the west coast of Sweden, from Guthenburg to Helsingbörg over the Strait of Oresund to Helsingör, and along the coast of Den-

ESTIMATED COST OF GENERATING AND TRANSMITTING ELECTRIC POWER; TROLHATTAN-COPENHAGEN PROJECT

	Alternating Current		Direct Current	
	Cost of Installation	Annual Expenses	Cost of Installation	Annual Expenses
Generating station:—				
Two 10,000-hp. turbines and auxiliaries	$81,000	$7,600		
Three-phase generators	70,500	6,600		
Transformers at Trollhättan	31,500	2,950		
Switch gear	27,000	2,650		
Wages, oil and supplies	.....	3,200		
	$210,000	$23,000		
Changes for turbines			$40,500	$3,400
Isolating machinery			2,700	200
			$43,200	$3,600
Four 5000-hp. turbines			$81,000	$7,600
Direct-current generators and auxiliaries			216,000	21,400
Wages, oil and supplies			.....	3,200
			$297,000	$32,200
Transmission line:—				
Iron poles, including erection, at at $1350 per mile	$275,000	$27,200		
Insulators, at $390 per mile	80,000	7,900		
Iron cable, at $87 per mile	17,900	2,500		
Copper conductors, at $1525 per mile	310,100	21,000		
Wages for linemen	.....	2,200		
	$684,000	$60,800		
Poles, including erection			$125,100	$13,000
Insulators			31,900	3,200
Iron cable			17,800	2,500
Copper conductors			166,200	11,300
Wages for linemen			.....	2,200
			$341,000	$32,200
Three transformer stations:—				
Buildings	$40,500	$3,400		
Transformers	81,000	7,600		
Instruments	30,500	3,000		
Auxiliaries	40,500	4,200		
Wages and supplies	.....	4,000		
	$192,500	$22,200		
Submarine cables:—				
Four cables, each 3.4 miles, at $5780 per mile	$78,600			
Loading on steamer, laying and splicing	20,000	$14,200		
Freight and marine insurance	5,400			
Anchorage at cables	9,500			
	$113,500	$14,200		
Two cables, each 3.4 miles, 150,000 circ. mils			$24,600	$5,400
Laying of cables, etc.			18,900	
			$43,500	$5,400
Frequency-converter station:				
Three frequency changers, 25 cycles to 50 cycles; total 11,600 kw	$122,000	$11,400		
Switch gear	13,500	1,300		
Wages, oil and supplies	.....	3,200		
Incidentals	27,000	3,200		
	$162,500	$19,000		
Direct-current motors and auxiliaries			$221,000	$22,000
Three-phase generators			51,500	4,800
Switchboard and switch gear			10,800	1,100
Wages and supplies			.....	3,200
Incidentals			54,000	6,200
			$337,300	$37,300
Incidentals	$231,500	$24,100	$140,000	$14,400
Total	$1,593,000	$163,300	$1,202,000	$125,100
Total per kw	.....	$15.66	.....	$11.16

mark to Copenhagen, a total length of 204 miles. The greatest natural obstruction met with along this route is at the Strait of Oresund, where a stretch of water is encountered of 3.4 miles, making resort to submarine cables necessary.

Two schemes were worked out, one covering the use of three-phase equipment and lines, and the other direct current at 90,000 volts, under the Thury constant-current system. In the first case, was a 100,000-volt line with 150,000-circ.mil. conductors in a single three-wire circuit on iron poles with suspension-type insulators. At Helsingborg it would be necessary to step down to 20,000 volts and carry the current in four three-phase submarine cables. From the Danish side of the strait the transmission could be continued at 20,000 volts, or at 100,000 volts, the latter being more economical. The main features of the estimate for this scheme are shown in the accompanying table.

The great advantage of the direct-current system was found to lie in the ability to transmit current under water at 90,000 volts without transformer stations at Helsingborg and Helsingör. Over against this advantage would have to be set the necessity of 20 direct-current generators in series. The generators would be arranged in four groups, each of five in series, and each group connected to a 5000-hp. turbine. The efficiency of the direct-current generators and motors was assumed to be 93%, compared with 95% average for the alternating-current generators and motors. A single transmission line would be used with a ground return. The transmission wire, however, would be split into two parallel sections, each of 119,000-cir.mil. cross-section. Both steel-lattice and impregnated-wood poles were compared, and the latter found more economical in first cost and annual expense. The total annual expenditures are estimated to be $6.75 per kw. for direct current and $11.16 per kw. for cycle alternating current, with 12,615 kw. available in the first case and 11,200 kw. in the second.

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Reservoir and Canal Losses in Irrigation

SYNOPSIS—One of the supervising engineers of the United States Reclamation Service has compiled valuable data on evaporation and seepage losses from four large reservoirs and on the total losses from reservoirs, main ditches and laterals of six irrigation projects. All the data are for the northwestern part of the United States. Canal linings and the use of pipe to prevent the loss of irrigation water are described. Figures are given to show the ultimate economy of lined as compared with valued canals.

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At the Seattle meeting of the American Society of Civil Engineers, there was read on June 27, 1912, a paper entitled "The Economic Aspects of Seepage and Other Losses in Irrigation" by E. G. Hopson, supervising engineer U. S. Reclamation Service, Portland, Ore. The paper has since been published in the *Proceedings* of the Society, Vol. XXXVIII, No. 8, p. 1223, where it was scheduled for discussion on Dec. 4, 1912. The article which follows is based on a copy of the original Seattle paper and illustrations supplied us by Mr. Hopson. The introductory portion of the paper has been slightly condensed, a few of the photographs have been omitted, and the tables and illustrations have been grouped for the sake of greater clearness and convenience.

Mr. Hopson's paper opens with a reference to data on the waste of water in city water-works systems, given in the report made in 1900 by John R. Freeman on the water-supply of New York City. Mr. Hopson points out that the arguments for cutting down waste in a water-works system apply with even more force to an irrigation system, since the economic burden of irrigation upon land under cultivation is much greater than that of domestic water-supply upon city dwellers, and since, too, it often happens that every bit of irrigation water wasted means just that much less land that can be made productive.

Having introduced his subject by this comparison, Mr. Hopson states that irrigation water wastes, like those from city water-works systems, may be divided into curable and incurable classes and that the object of his paper is to discuss these two classes of wastes, as illustrated by government irrigation works built in the Northwest during the past half-dozen years.

Broadly speaking, incurable irrigation losses are those due to absorption in reservoir beds and to evaporation into the air, while curable losses are confined almost wholly to seepage from canals and losses from other conduits.

TABLE I. WATER LOSSES FROM FOUR IRRIGATION RESERVOIRS, NORTHWESTERN UNITED STATES

East Park Reservoir, 1910-1911

(Maximum capacity, 45,000 acre-feet. Maximum water surface, 1690 acres)

	Acre-feet	Percent. of influent
Influent	65,400	
Effluent and losses:		
Evaporation	7,100	11
Use, waste and surplus	58,300	89
*Seepage	0	0

*No appreciable seepage loss.

Cold Springs Reservoir, 1908-11.

(Maximum capacity, 50,000 acre-feet. Maximum water surface, 1550 acres.)

	1908		1908–9		1909–10		1910–11	
	Acre-feet	Per cent.	Acre-feet	Per cent.	Acre-feet	Per cent.	Acre-feet	Per cent.
Influent	20,366		42,820		61,526		72,273	
Effluent & losses								
Evaporation	2,400	12	4,295	10	5,333	9	6,252	9
Seepage	4,515	22	*4,021	9	*10,461	17	10,878	15
Use, waste and surplus	13,451	66	34,504	81	45,732	74	55,163	76
*Return flow			865		503		182	

Clear Lake Reservoir, 1909-11

(Maximum capacity, 450,000 acre-feet. Maximum water area, 25,000 acres.)

	1909–10		1910–11	
	Acre-feet	Percent.	Acre-feet	Percent.
Influent	141,000		225,000	
Effluent and losses:				
Evaporation	80,000	57	88,000	39
Seepage	48,000	34	24,000	11
Use, waste and surplus	13,000	9	113,000	50

Deer Flat Reservoir, 1909-11

(Maximum capacity, 186,000 acre-feet. Maximum water area, 9250 acres.)

	1909		1909–10		1910–11	
	Acre-feet	Percent.	Acre-feet	Percent.	Acre-feet	Percent.
Influent	64,000		130,000		230,000	
Effluent:						
Evaporation	4,000	6	18,000	14	20,000	9
Seepage	55,000	86	80,000	62	140,000	61
Use, waste and surplus	5,000	8	32,000	24	70,000	30

The percents. in each case are percentages of volume of influent.

Water Losses from Irrigation Reservoirs

It is an economic impossibility for an engineer to prevent absorption or seepage losses from the beds of irrigation reservoirs, but he must accept responsibility for the choice of reservoir sites which will be a factor in reservoir losses. On this account Mr. Hopson presented data for four typical irrigation reservoirs built by the federal government in the Northwest. We quote from his paper as follows:

THE EAST PARK RESERVOIR is strictly a storage reservoir built on a branch of Stony Creek, one of the Coast Range feeders of the Sacramento River. The dam site is a good one, being a notch in a great conglomerate dike or ridge that runs through the country in a north and south direction, and

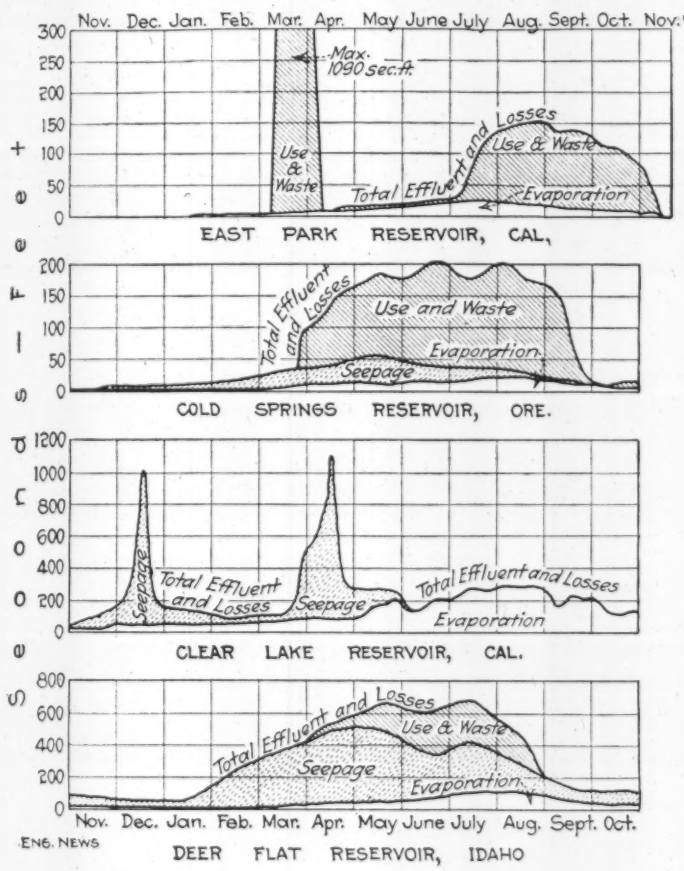


Fig. 1. Monthly Records in Second-feet, 1910-1911

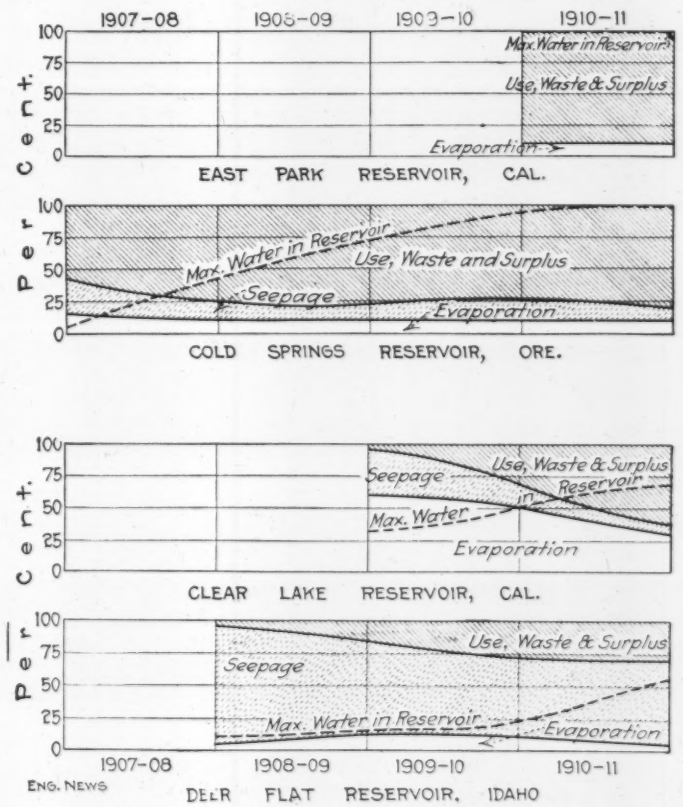


Fig. 2. Percentages for Four Years

Figs. 1 and 2. Evaporation, Seepage, Use and Waste of Water for Four Government Irrigation Reservoirs in the Northwestern United States

This reservoir was first placed in commission in the Spring of 1908, and has been operated ever since. We have, therefore, four yearly records of results. In this case measurements were obtained with unusual accuracy, as the inflow practically all passed over a sharp crested weir at the lower end of the feed canal and the effluent was also carefully measured over another weir below the outlet gates. This reservoir shows losses ranging from 34% to 24% of the influent during the four-year period. Judging by the record of the past two years, it would appear that a fair condition of stability has been attained in the regimen, in which about one-fourth of the water entering this reservoir is subject to unavoidable loss through seepage and evaporation. (See Table I and Figs. 1 and 2.)

THE CLEAR LAKE RESERVOIR in California is a feature of the Klamath Project, situated just south of the California-Oregon line. It occupies a great natural depression or sink some 25,000 acres in extent at the reservoir flow line. About one-half of the bed consists of a natural sink of alkaline water known as Clear Lake that has for ages received and evaporated the surplus waters of Willow Creek. This reservoir was built by the Government for the principal purpose of holding back the waters of Willow Creek, to facilitate the unwatering of land marginal to Tule Lake, a body of water into which Willow Creek ultimately discharges. The reservoir was intended to combine the purposes of a great evaporating pan and regulator of the diversion channel that diverts the discharge of Lost River from Tule Lake into Klamath River. More recent plans have, however, considered its possibilities as a source of irrigation supply. The reservoir is enormous in capacity as compared with the available water-supply, being 450,000 acre-feet, with an area of 25,000 acres. The dam on Willow Creek is a rock-fill structure some 30 ft. in height, completed in 1909. We have two years records of the action of this reservoir. (See Table I. and Figs. 1 and 2.)

The rate of evaporation in this vicinity has been estimated at a little more than four feet in an average year. It will be noted that evaporation is the principal loss in this reservoir, as had been anticipated. Seepage losses during the first year were heavy, but apparently the marginal lands became filled up so that losses in 1911 were comparatively moderate. It is important to note that in a year of copious runoff, like 1910-11, as much as 50% of the supply was subject to unavoidable loss or waste, which in this case was intentional, the principal purpose of the reservoir being the disposition of surplus water, rather than its conservation for use.

THE DEER FLAT RESERVOIR, a feature of the Boise Project, in Idaho, presents different natural conditions from the three preceding types. It does not occupy a natural drain-

the dam is a solid masonry structure of gravity type on an arched plan. The bed of the reservoir is practically wholly in the typical California shale. The dam was completed in 1910 and water first stored in the winter and spring of 1910 and 1911. Weekly measurements are taken of the influent and effluent, the storage and rates of evaporation.

The maximum capacity of the reservoir is 45,000 acre-feet, and maximum area of water surface 1690 acres. In Table I may be found the results in the 1910-1911 season, the season being from Nov. 1 to Nov. 1 in this and all following cases. (See also diagrams Figs. 1 and 2.)

This reservoir represents the highest condition of efficiency of any of the four herein illustrated. The records fail to show any seepage loss, the only appreciable loss being that by evaporation, thus nearly 90% of the water entering this reservoir being available for use.

THE COLD SPRINGS RESERVOIR of the Umatilla Project in Oregon is a good average reservoir from a western standpoint. In the East it would probably not be regarded as a site of special promise. The dam is an earthen one, nearly 4000 ft. long, of a maximum height of close to 100 ft. The general structure of the country is volcanic, with vast overlying beds of stratified sands, gravels and hardpan. The valley constituting the reservoir site is the outlet of some 200 sq. miles of drainage area with little or no ordinary runoff. The reservoir is supplied by a feed canal some 25 miles long diverting from the Umatilla River at times when the latter has available water.

age valley or sink, being, on the contrary, situated on a flat saddle between the hills, the lower ends of which are closed by two earthen dams. The reservoir derives its supply, as in the case of the Cold Springs Reservoir, through a feeder canal, known as the New York Canal, diverting from the Boise River some ten miles southeast of Boise. The reservoir was first placed in commission in 1909, and has been in operation ever since. The bed of the reservoir consists in large part of silts, sands and gravels, with a covering of from 3 to 5 ft. of soil. Seepage losses in this case have been pronounced from the outset, and constitute the bulk of all losses. When the reservoir was first placed in commission almost 90% of the water entering it was lost by absorption in the reservoir bed. In that year, however, the reservoir was only filled to one-tenth of its capacity. During the next two seasons larger and larger quantities were introduced, and the proportion of losses has appreciably fallen, but still remains high. During the last season about two-thirds of the water entering this reservoir was subject to loss through evaporation and seepage. It may be expected that conditions will improve at this point, as the adjacent and underlying strata of the reservoir gradually become filled by the constant application of water, but the extent and period of these ameliorating conditions are quite uncertain. (See Table I and Figs. 1 and 2.)

The above records, while incomplete and faulty in many respects, are among the best obtainable in a new country like this, and in any event are instructive. I think the general problem of reservoir losses is often given less attention by engineers than its importance warrants. The dam site is apt in many cases to monopolize attention, and an engineer accustomed to deal with reservoir sites in eastern river valleys, where the adjacent water tables are high and losses are almost confined to evaporation, may be led to the commission of grave mistakes.

A great deal has been said and written about return flow. One of my earliest recollections in connection with reservoir studies is the discussions in the proceedings of the Society between Messrs. FitzGerald Stearns, Fteley and others on ground water storage of certain reservoirs in the East. Mr. FitzGerald's conclusions as to the general inadvisability of giving credit to the invisible storage of a reservoir, is I believe, wise. Save under exceptional conditions, I doubt whether much, if any, additional draft can be made from Western reservoirs in excess of the visible storage.

The Cold Springs Reservoir has, during the past four years, absorbed some 30,000 acre-feet of water in its bed. It has apparently only yielded back about 1500 acre-feet of this amount. The Deer Flat reservoir has apparently absorbed 270,000 acre-feet, with little or no return.

It should be observed that the amount of water discharged into Deer Flat Reservoir has largely increased each succeeding year, thus submerging large areas of new surface and that in spite of this fact, the percentage of loss is steadily decreasing. This decrease of loss has extended into the year 1912, of which complete records were not yet available when this paper was written.

It is important to note that in a reasonably good representative irrigation reservoir, such as Cold Springs, one-quarter of the water turned into it is lost, and that apparently under the most favorable circumstances, as at East Park, 10% will be lost.

The main lesson to be derived from these few illustrations is that the geologic structure of the site should be given the most careful consideration, it being vital to determine in advance as nearly as may be the amount of reservoir losses, and whether they are likely to be of a permanent character.

On the Umatilla Project the cost of the irrigation works per acre of irrigable land is from $60 to $70; on the Truckee-Carson Project about $40; on the Orland Project about $50; the Tieton about $90; Sunnyside about $50, and Klamath from $30 to $40, say an average of about $55. This is a fair indication of the general run of costs in large irrigation work hereabouts, and is probably lower than the average costs on newer projects either government or private.

LOSSES FROM RESERVOIRS, CANALS AND FARMERS' DITCHES

The various losses in the water-supply system as expressed in percentages of the water diverted are shown in Table II.

The losses from the diversions down to the end of the regular lateral systems that are operated by the Government, run from one-fourth to upwards of one-half the whole supply, but below these are the ramifications of the small ditches built by the farmers to distribute water to their farms. These farm ditches are usually small earth trenches, in which heavy seepage occurs before the water actually reaches the crop. Although in some cases farmers use water-tight flumes and pipes for their local distribution, the proportion of these cases is as yet comparatively small, although on the increase. It has been estimated that seepage losses in the farmers' ditches on many projects is not less than 50% of the losses in main canal and lateral systems. It should be understood that the losses in the farmers' ditches (Table II.) are merely the expression of individual opinion, not of actual measurement.

TABLE II. TOTAL WATER LOSSES FROM SIX IRRIGATION PROJECTS

	Reservoir	Canal Losses: Canals and laterals	Canal Losses: Farmers' ditches	Total: Without farmers' ditches	Total: With farmers' ditches
Umatilla	20	32	15	52	67
Truckee-Carson	0	41	15	41	56
Orland	8	23	10	31	41
Klamath	0	48	15	48	63
Tieton	0	24	8	24	32
Sunnyside	0	27	7	27	34

[Fig. 3 shows volumes of water diverted to six irrigation projects in the irrigation season of 1911 and what became of the water—Ed.]

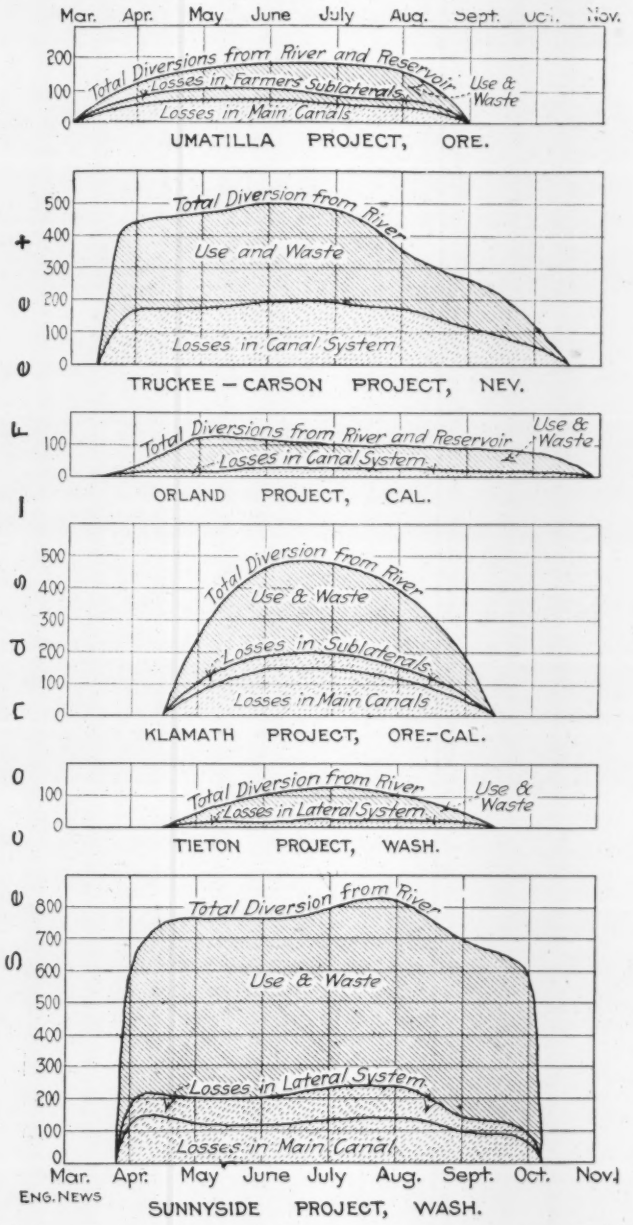


FIG. 3. TOTAL RIVER DIVERSION, USE AND WASTE LOSSES IN SUBLATERALS AND IN MAIN CANALS ON SIX IRRIGATION PROJECTS IN THE NORTHWESTERN UNITED STATES, SEASON OF 1911

LINING CANALS TO PREVENT SEEPAGE LOSSES

Seepage losses on the Umatilla Project early assumed serious proportions, owing to the sandy character of the soil and the gravelly substrata. With the unlined earth ditches as originally constructed only about one-third of the water diverted reached its proper destination. The works were costly and the quantity of supply limited. Unless means could be found to lessen these losses it was evident the entire area could not be irrigated and the building costs would not be wholly repaid.

Other severe proportional losses were found on the Truckee-Carson Project in Nevada and the Klamath Project in Oregon,

but in both of these latter is more elasticity due to their greater available supplies and, in the case of the Klamath Project, to the small aggregate quantity used.

Losses on the Tieton and Sunnyside Projects are probably much more satisfactory than in the average well constructed project in this vicinity, due in the first cast to the complete concrete lining of the main canal and the tight character of the substrata of the irrigated lands. In the Sunnyside Project the relatively small canal losses are due mainly to the fine texture of the soils.

It is probably a fact that in the average project from 40 to 50% of the supply is lost by seepage in the beds of channels before it reaches the actual point of application. As the farmer is paying from $30 to $90 per acre for this water the loss has a very appreciable value.

In Southern California valuable orchard lands have been under irigation for a generation. Corp values have been very high and, in many cases, the water-supply has been so limited that effective measures toward conservation have been enforced. On many projects there the distributing channels are lined with concrete or pipe is used with liberality. The high values of lands and the scanty supply have rendered these measures not only desirable but necessary. Strict economy in use has also been enforced for the same reason, but in the newer projects in the Northwest, where lower valued crops obtain, it has not hitherto been seriously regarded as feasible to resort to such expensive treatment. Conditions have, however, materially changed here with regard to crop values, and many of the water-supplies that appeared inexhaustible a few years ago are rapidly getting fully appropriated, so that reasons for economy and waste prevention are becoming more and more cogent.

Some interesting experiments* carried out under the auspices of the College of Agriculture of the University of California, in 1906, by Prof. B. A. Etcheverry, on various kinds of canal linings, including concrete, clay puddle and oiled surfaces, are worthy of consideration. The object of this experiment was to determine relative costs and efficiencies of different classes of lining in reducing seepage and preventing growth of vegetation. Without attempting to enter into details of these experiments, which can be obtained in the University Bulletin*, the general results showed that the concrete lining, although the most expensive, alone gave assured results. The oiling, as would be expected, is very much cheaper than any other treatment, being only about one-quarter the cost of the concrete per square foot. During the first year it appears to be of some value in reducing seepage losses, measurements showing that losses as compared with those in an untreated earth canal are only about 40% of the latter. The oil seemed to be principally valuable in preventing growth of vegetation. The lining by clay puddle gave somewhat better results in preventing seepage than the oiled surface. The mortar and concrete linings, however, prevented from two-thirds to nine-tenths of the total losses and, of course, entirely stopped the growth of vegetation.

Apparently the effect of the oil treatment is only temporary, and a re-examination of the canals in which the experiments were made a year or two afterwards showed that the oil treated canals had grown up to vegetation to an equal extent to those untreated, and in all probability the seepage losses were equally as great.

*See Eng. News, Dec. 5, 1907, pp. 620-2, for abstract.

In 1910 and 1911 a lateral on the Umatilla Project was lined with mortar 1 in. thick and careful measurements made to determine losses. The lateral had been selected for lining on account of the very porous character of its bed and in order to reduce seepage loss. With the lateral closed at the ends by dams, measurements showed that the water surface lowered about 0.1 ft. each day in the lined ditch, and applying this rate of loss to the canal system as a whole, making due allowance for velocity of flow, it was computed that the aggregate seepage loss in the project, if all canals were lined, would be about 5% of the supply. With the unlined system the loss is close to 50%. Other measurements that have followed confirm the above, and the conclusion has been reached from this and other data that seepage losses can be kept down to less than 10% of the amount diverted if good linings are placed.

[Two mortar-lined laterals and one unlined farmer's ditch on the Umatilla Project are shown by Figs. 4 and 6 and a concrete-lined canal on the Tieton Project by Fig. 5.—Ed.]

During the past two years a great deal of canal and ditch lining has been placed on government projects in the Northwest. These linings are from 4 in. to 1 in. in thickness, depending on the size of the canal and the conditions. The heavy lining is made of regular sand and gravel concrete proportioned about one cement to eight of sand and gravel. The linings are generally placed without forms, the sides of the channel being trued up and a rather dry mix being used. Costs have usually run about $6 per yard. The great bulk of the ditch lining has not, however, been of regular gravel concrete but of mortar, usually being mixed one of cement to four of sand. Before placing the mortar the ditches are carefully trued up by running a movable form or templet along their courses and wetting and tamping the earth around the form. Immediately after the form is removed the mortar is placed and kept damp until well set up. It is jointed usually at about 4-ft. intervals to take care of temperature shrinkage. This lining is done with much rapidity by experienced gangs, the materials being mixed by small portable gasoline-driven mixers, the completed canals being kept full of water. Costs of work of this kind carried out on a fairly large scale—for lining 1½ in. thick, reinforced at the top by an extra heavy curbing—runs from 55 to 60c. per sq.yd., inclusive of all administrative and engineering charges and of the earthwork. In general, the cost of the earthwork is about one-third of the entire cost.

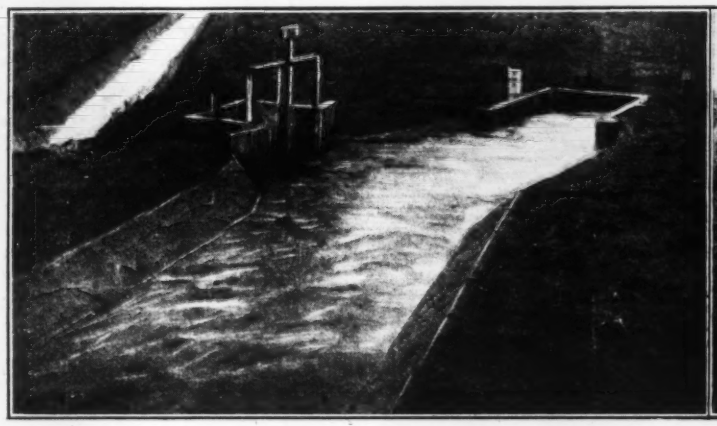


FIG. 4. MORTAR-LINED LATERALS AND CONCRETE STRUCTURES

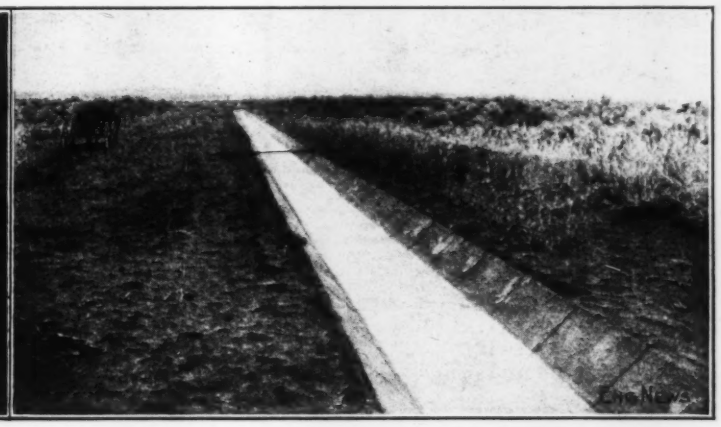


FIG. 5. MORTAR-LINED LATERAL, SHOWING JOINTS

FIGS. 4 AND 5. LINED IRRIGATION CANALS ON THE UMATILLA PROJECT

Taking for example a small lateral of the Umatilla Project lined in this way during 1911: the length was 12,400 ft.; ditch dimensions 4 ft. wide at bottom, 4 ft. deep, with side slopes 1½ to 1; the entire cost of the work averaged $1.05 per ft. Comparing a small ditch thus lined with one unlined, the lined ditch will be from three to four times the cost of the unlined ditch, but one of the economic advantages the lined ditch possesses is the greater velocity of flow possible and the consequently smaller cross-sectional area of the channel. Another advantage of importance is the avoidance of drop structures possible in the lined ditches. It is surprising what a large proportion of the total cost of ditch building goes into drop structures that are necessary to keep velocities below the eroding point in an earth channel. With lined channels high velocities are not only possible but desirable in order to keep the channel clean.

As an illustration of the proportion the cost of structures in a distribution system bears to the entire expense of the latter, figures taken from the Orland Project in California are given. The territory here is notably free from topographic irregulari-

ties and the earth is firm and good for building purposes. The proportion of cost of structures here, therefore, would be expected to be small.

The lateral system covers 14,000 acres and includes 54 miles of ditches ranging from 12 to 75 sec.-ft. capacities. Very little ditch lining has been placed, but structures are all of concrete, the cheapest building material. Cost totals are as follows:

Excavation work	$64,376
Structures	57,632
Total	$122,008

The structures included in the above comprise the following types:

Checks and drops	$20,885
Turnouts	12,901
Bridges	9,972
Railroad crossings	6,924
Special structures	5,805
Spillways	1,143

Checks, drops and turnouts total $33,780, or 28% of the entire cost of the lateral system. Not all of this cost could be obviated by lining the system, but certainly a very large proportion of it would be.

With a smaller cross-sectional area, the saving in drop structures, and the more direct and economical location possible in the lined ditches, the actual difference in cost per acre of land served by lined or unlined canals is comparatively small. It will generally be found less than $10 per acre, in many cases less than $5. If one takes into consideration the operating economies, the lined laterals have a distinct advantage in their freedom from breaks, seeped banks, and growth of vegetation in the channels, all of which should admit of a material reduction in operating costs. If these latter savings could be figured from an investment standpoint and capitalized, the advantage in first cost of the unlined ditches would often disappear and a substantial margin be shown on the other side.

PIPE FOR DISTRIBUTION SYSTEMS

While considering canal lining it would be well to pay a little attention to the merits of pipe work in a distributing system. Large quantities of pipe work have been used in the distributing systems of the Umatilla, Tieton and Sunnyside Projects. The great bulk of this pipe is concrete, both reinforced and plain, in sizes running from 54 in. down to 12 in. in diameter. Sizes below 24 in. have been usually made by the dry process, reinforcement consisting of outside wire, wound under tension. Larger diameters have usually been wet mixed, the pipe being manufactured in yards and hauled and laid like cast-iron pipe. Some of these lines of pipe are of great length and are working under heads running up to 110 ft. They have uniformly given satisfaction from every standpoint. A distribution system consisting wholly of concrete pipe would undoubtedly be the most satisfactory from an operating standpoint, and although the first cost would be comparatively high, it might in the end prove to be more truly economical than the open ditch systems. With concrete pipe seepage losses are practically negligible. A number of tests of different lines of 4-ft. pipe, with 3-in. shell, under operating conditions, show the following:

Length, ft.	Head, ft.	Average seepage, second-feet per mile
4700	39	0.07
5400	28	0.05
3600	19	0.04
9800	85	0.20

Apparently the loss per mile of this size pipe is nearly directly proportional to the head, averaging about 0.02 sec.-ft. per mile for each 10 ft. head carried on the pipe. A pipe distributing system of concrete throughout, under an average pressure of 50 ft. head, with delivery to each 40-acre subdivision, would thus only lose about 1% in amount, which is practically negligible.

Taking the average of the six projects quoted, we have an average cost of the irrigation works of $55 per acre and an average combined loss in reservoirs and canals of about 50% of the entire water-supply. Of the latter about 6% is practically incurable reservoir loss; the remaining 44% has been classed as curable; that is to say, the great bulk of it can be cured or prevented if economical conditions render such action wise.

Should the ditch systems of these projects be wholly lined with concrete or pipe the losses might be reduced from 44% to 10% or less in amount, a net saving of 34%, or say one-third of the whole supply. It is therefore evident that either the systems could be extended to cover about one-third more area, or should that land not be available, the works might be constructed of smaller dimensions and at less cost. In the case of works already built the latter alternative is inapplicable and is merely illustrative of what might have been done but what cannot be helped now. The lesson should, however, be applied to new work. In cases, however, where new lands can be taken in under existing works consideration should be given to the possibilities of extension by lining the present systems.

ADVANTAGES OF CANAL LININGS AND OF PIPE

Suppose, for example, a project of 20,000 acres costing $55 per acre, or a total of $1,100,000; if by lining the ditches the irrigable area can be increased to 27,000 acres there will be first an additional cost for the new laterals with lining which has been found to be somewhere about $18 per acre in a fairly difficult country, or for the 7000 acres an additional construction cost of $126,000 will be necessary. Secondly, there will be the cost of lining the present ditch system covering 20,000 acres, which, taken at $12 per acre, would mean an added charge of $224,000. The gross cost of the extended project would, therefore, be $1,450,000, or an average cost of $54 per acre. This does not apparently result in a material reduction in the acreage cost, but the great advantage lies in rendering available for profitable use the larger areas of land, the conservation of the water-supply and the avoidance of drainage evils referred to hereafter. As a matter of fact, the process of extending a project already constructed with unlined earth canals, by the subsequent lining of ditches, is always much more expensive than if the system had been constructed de novo with all of the system lined.

In the case of a proposed large extension of the Umatilla

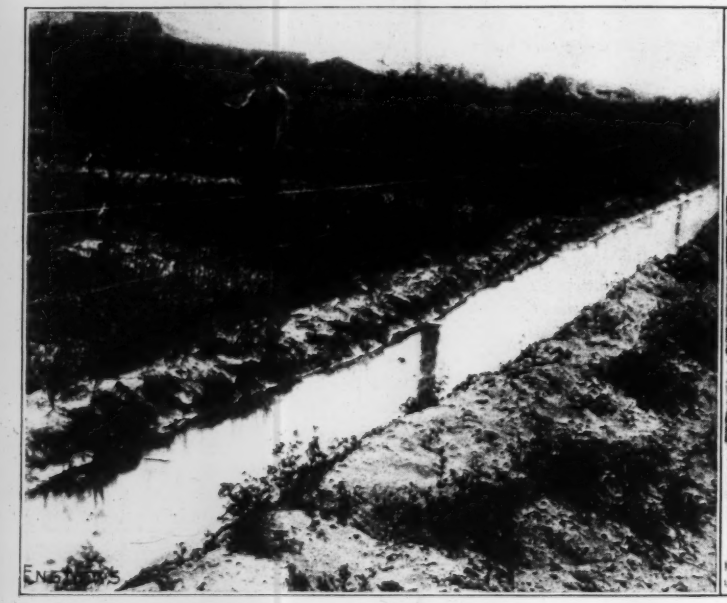


FIG. 6. TYPICAL FARMER'S IRRIGATION DITCH, UMATILLA PROJECT

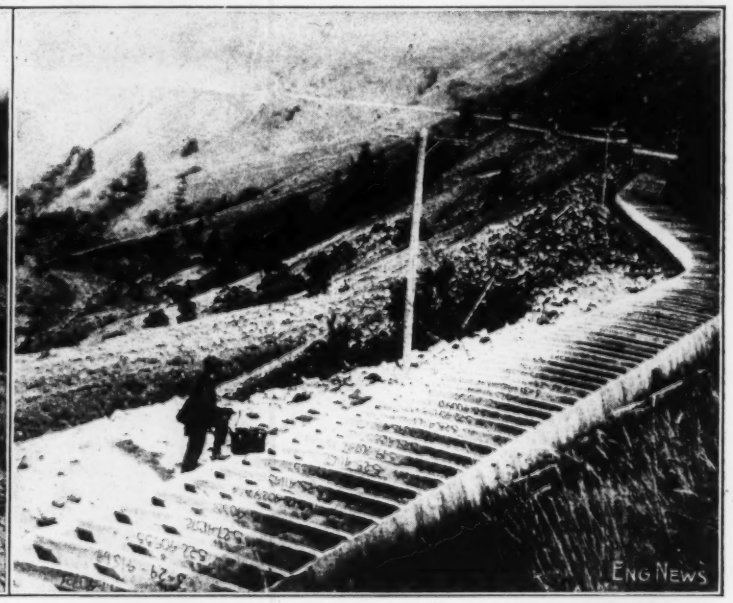


FIG. 7. CONCRETE-LINED MAIN IRRIGATION CANAL, TIETON PROJECT

Project it is planned to completely line the entire canal system from the headworks down to the minor ramifications of the distribution system delivering to each 40-acre subdivision. At no point in the system will the water be exposed to avoidable seepage loss, and when the headgates at the reservoir are opened the government will have assurance that upwards of 90% of the supply will actually reach the cultivated fields.

THE DRAINAGE PROBLEM

Closely connected with the question of canal losses is the drainage problem. On nearly every irrigation project there will be found large and frequently increasing areas subject to the rise of ground water. The principal contributing influence in most cases is the seepage losses from the lateral systems, although a proportion, of course, is due to over irrigation of the fields.

On the Sunnyside Project in Washington some four or five thousand acres of the best land was seriously affected when the United States purchased the system a few years ago, large areas having been practically forced out of cultivation. In this case the United States was compelled to build a deep channel at a cost of some $340,000, mainly for the purpose of affording an outlet to the surplus water. On the Minidoka Project in Idaho the drainage feature is one of the most serious problems.

At Umatilla the seepage water accumulating below the project in the Umatilla River has increased the summer flow some 100 sec.-ft. and has rendered necessary the excavation of extensive drainage ditches through the lower lands.

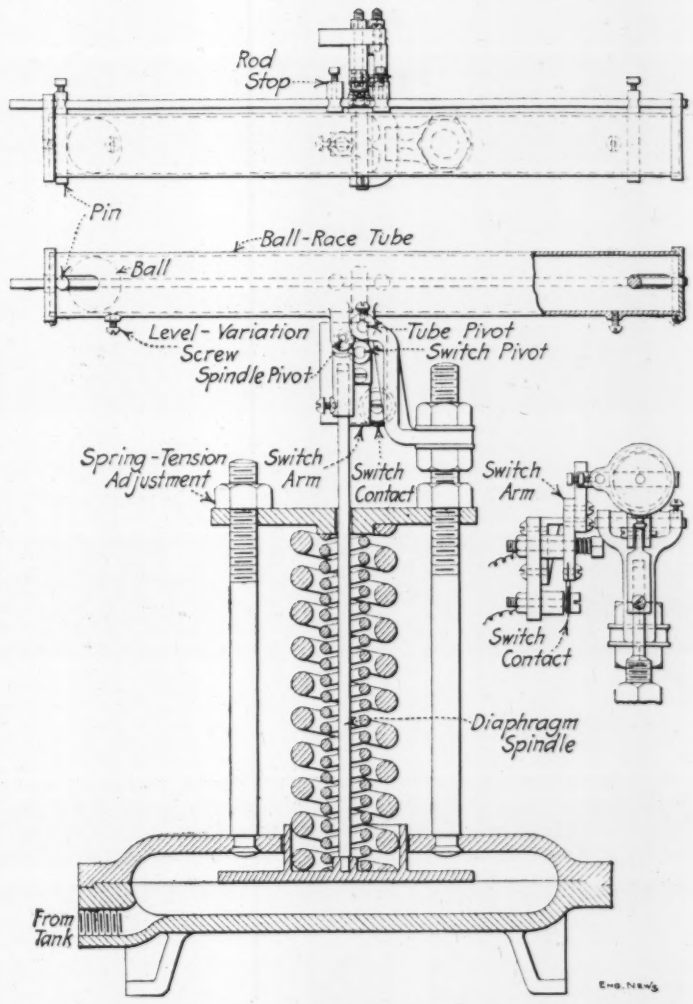
At Klamath some $40,000 has been spent during the past three years on this account, and on the Truckee-Carson Project it is planned to expend not less than $400,000 in addition to the large sums already spent.

There is no question that much relief from this increasing danger will be experienced by eliminating from the ground water accumulations the bulk of the canal seepage. It is my belief that as time goes on it may even be found necessary for legislatures to require canal systems to be lined or otherwise protected from seepage loss not only in the interests of the investor and water user but as a reasonable measure of conservation when water-supplies are limited. As an engineering and business policy it is well in the front rank for consideration by all those building new works or operating and extending those already constructed.

A Pressure-operated Switch for Pump Motors and Low-water Alarms

The electric switch shown in the accompanying figure has been designed to serve the purpose of the more cumbersome float-operated switches which require a considerable travel of the float mechanism and are subject to such mechanical difficulties as sticking, freezing, etc. Pressure from the receiving tank is transmitted to the chamber under the diaphragm, forcing this, with its attached rod, upward and tilting the horizontal hollow cylinder at the top of the device. This movement of the cylinder permits a large ball inside to roll from the left-hand end to the right, striking a pin which projects into the interior of the cylinder and which is attached to a rod moving parallel to the cylinder. This motion is transmitted through a stop on the rod to the switch arm, opening the electric circuit and stopping the pumping motor.

When the water level drops, and with it the pressure under the diaphragm, the springs about the diaphragm spindle force the latter down and return the cylinder to its horizontal position, whereupon the ball rolls from right to left, striking the corresponding pin projecting into the cylinder at the end, and moving the parallel rod so that the stop on it throws the switch lever into contact to complete the motor or alarm circuit. The inclination of the horizontal tube, which is necessary before the ball is released, can be adjusted by the screws shown on the under side. This controls the variations of water level in the water tank; actual height of water level is varied by changing the tension on the springs of the diaphragm



DIAPHRAGM TYPE OF ELECTRIC SWITCH FOR PUMP MOTORS OR LOW-WATER ALARMS
(Golden Anderson Valve Specialty Co., Pittsburgh)

spindle, and for this two frame rods are provided with adjustment nuts and a plate bearing on the springs. The device is built by the Golden-Anderson Valve Specialty Co., Fulton Bldg., Pittsburgh.

Public Ownership of Telephones in Canada has reached considerable proportions. A pamphlet report, giving "telephone statistics" of Canada for the year ending June 30, 1912 (A. W. Campbell, Deputy Minister, Department of Railways and Canals), states that in Manitoba and Alberta all telephone companies have been practically absorbed by the provincial governments, while in Saskatchewan similar absorption is now in progress. The number of municipal telephone systems owned by the various provinces is as follows: Ontario, 27; Alberta, Saskatchewan, Manitoba and British Columbia, 8 altogether, making 35 municipal telephone systems in the whole Dominion. There are also in the Dominion 133 coöperative, 31 partnership, and 113 private telephone systems against 368 stock companies and the public systems already mentioned.

Electrically Operated Excavating Machinery is to be used in the construction of the diversion channel of the Little River Drainage District, in Missouri. Current will be generated at a central power station, transmission lines being run along the channel. The excavators will be revolving machines handling clam-shell and drag-line buckets, and traveling on tracks laid along the work. The excavated material will be used to form levees. This channel will discharge into the Mississippi River about two miles below Cape Girardeau, Mo. It will intercept the Castor and Whitewater Rivers, Crooked Creek, and other streams whose present outlet is through Little River to the St. Francis, and then into the Mississippi River, near Helena, Ark. The contractor for this work is D. C. Stevens, of Buffalo, N. Y., whose bid for the two sections amounted to about $1,229,585. This covers 3907 acres of clearing and 8,621,500 cu.yd. of excavation. The right-of-way has been obtained and the clearing of the timber will be commenced shortly. The district has an area of about 500,000 acres.

A Comparative Test of Two Full-sized Reinforced-concrete Flat-slab Panels

By Henry T. Eddy*

Two experimental reinforced-concrete flat slabs were submitted to test by Prof. W. H. Kavanaugh, of the University of Minnesota, at Minneapolis, during the latter part of 1912. The first slab was made in accordance with the specification of the Norcross patent† except that belts of rods were substituted for the netting mentioned in the each belt. The concrete was of 1:2:4 mix, and while only about four weeks old, had been poured warm and under such unusually favorable conditions as to have become well hardened at the time of test.

The astonishing difference in stiffness and the remarkable difference in strength of these two slabs, the sudden, complete and total collapse of the first slab, contrasted with the slow, gradual yielding of the second under an excessive load several times as great as that which caused the first to fail, present an object lesson in safe and economic design worthy of the most careful study.

Fig. 1. Norcross Slab

Fig. 2. Mushroom Slab

Figs. 1-2. Test Slabs before Concreting, Showing Reinforcement Arrangement

specifications. The second slab was made according to the Turner "mushroom" system.

These slabs were both of the same thickness, viz., 6 in., and were supported by columns placed at the corners of a square 12 ft. from center to center of columns. The slabs projected approximately 2½ ft. beyond the centers of the columns on each side and had each precisely the same number and size of reinforcing rods, in

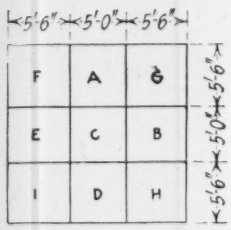


Fig. 3. Arrangement of Loading on Norcross Slab

TABLE I. LOADS ON NORCROSS SLAB IN POUNDS

Load No.	1		2		3		4	
Areas	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total
A	114.1	3,138	228.7	6,288	228.7	6,288	228.7	6,288
B	114.1	3,138	228.7	6,288	228.7	6,288	228.7	6,288
C	114.1	2,852	240.1	6,002	456.8	11,420	456.8	11,420
D	114.1	3,138	228.7	6,288	228.7	6,288	228.7	6,288
E	114.1	3,138	228.7	6,288	228.7	6,288	228.7	6,288
F							250	7,560
G	Corner Areas not loaded						250	7,560
H							250	7,560
I							250	7,560
Slab	60	15,404	121	31,154	143	36,572	276	66,812

*Professor of Mathematics and Mechanics, College of Engineering, and Dean of the Graduate School, Emeritus, University of Minnesota, Minneapolis, Minn.

†See Specification U. S. Patent No. 698,542.

Norcross Slab

Norcross, in his patent already referred to, describes his construction as consisting

> essentially, of a panel of concrete having metallic network encased therein, so as to radiate from the posts on which the floor rests. . . . . The posts are first erected, and a temporary staging built up level with the tops of posts. Strips of wire netting are then laid loosely in place on top of the staging. . . . . The concrete is then spread upon or molded in place on the staging to inclose the metallic network. In practice I have sometimes laid the concrete in layers of different quality, the lower layer of the floor which incloses the wire being laid with the best concrete available. . . . . If the forces acting upon a section of flooring supported betwen two posts be analyzed it will be found that the tendency of the floor section to sag between its supports will cause the lower layers of the flooring to be under tension while the upper layers of the flooring will be under compression, these stresses being, of course, the greatest at the top and bottom layers, respectively.

The number and arrangement of the reinforcing rods in the experimental slab (eleven ⅜-in. round rods in each side and diagonal belt), is clearly shown in Fig. 1, which is a view of the forms ready for pouring the concrete. It will be noticed that these rods lie very nearly upon the bottom of the forms throughout their entire length, both over the columns and elsewhere, and the main distance from the center of action of the steel to the top surface of the slab is taken as 5¾ in. This arrangement is in accordance with the Norcross specifications. Steel plates, 20x20x⅞ in., carry the rods and rest on the tops of the columns which, in this case, consisted of steel pipes about 5½ in. in diameter, filled with concrete and embedded at their lower ends in large concrete blocks.

A vertical central bolt in the concrete at the upper end of each pipe permitted the plates to be firmly secured to the tops of the columns. Fig. 4 shows the manner of loading the slab with pig iron. The slab is 16x16 ft. and the columns are on the corners of a 42-ft. square. The loading at first covered an area having the form of a Greek cross whose central square was 5 ft. on a side with arms 5 ft. 6 in. long, as represented in Fig. 3, and the progress of the loading is shown in Table I.

When 10,000 lb. had been piled on the central part of the slab in addition to load No. 4, of 66,812 lb., the slab suddenly failed. In anticipation of such failure, timber blocking had been placed under the slab to prevent its falling more than possibly 10 or 12 in.

Fig. 5 shows the condition of the slab after removing part of the final loading in order to render the nature of the failure visible. Careful extensometer measurements of the elongations of the steel rods at the middle of the side and diagonal belts were made under the action of loads Nos. 1, 2, 3 and 4, and also similar extensometer measurements in the concrete both on the top and the bottom of the slab along the center line of the side and diagonal belts near those edges of two of the steel plates which were nearest the center of the belts. Besides these, certain other measurements of the concrete were made at right angles to the diagonals. Deflections were also measured under these loads at the middle of the diagonal belt and of two of the side belts.

These measurements all show beyond question that the side and diagonal belts act like simple beams in this form of construction, since the stresses in the steel and concrete on the under side of the slab in the direction of the rods is invariably tensile, while the stresses in the same directions on top of the slab are always compressive. It was the avowed intention of Norcross to reinforce the slab in this manner since he regarded the upper part of the slab as being subjected everywhere to compression and the lower part to tension only, as stated in his specifications as already quoted.

The computation tabulated in Table II shows a good approximate agreement of the results of this test with the beam theory of flexure, assuming, for simplicity, that the stiff steel supporting plate and interlacing of the ends of the belts diminishes the effective span of the side belts by 12 in. and the diagonals in the same proportion, and further assuming that the loading was all applied at the middle of the side and diagonal belts.

The comparisons given in Table II were computed according to the following methods:

Extensometer measurements were made for a length of 8 in., consequently the stress per square inch in the steel is

$$f_s = \tfrac{1}{8}\, e \times 30{,}000{,}000 \qquad (1)$$

Where e = elongation in 8 in.

Then, using the following notation:

W = Load causing elongation e;
A = Total area steel;
d = Depth of slab to center of steel;
L = Length of span;
j = Ratio of lever arm of resisting couple to depth d;
id = Distance from center of steel to neutral axis.

Assuming a concentrated load W at the middle of the beam,

$$M = \tfrac{1}{4}\, W\, L$$

$$M = Ajdf_s \qquad (3)$$

$$W = \frac{4Ajdf_s}{L} \qquad (4)$$

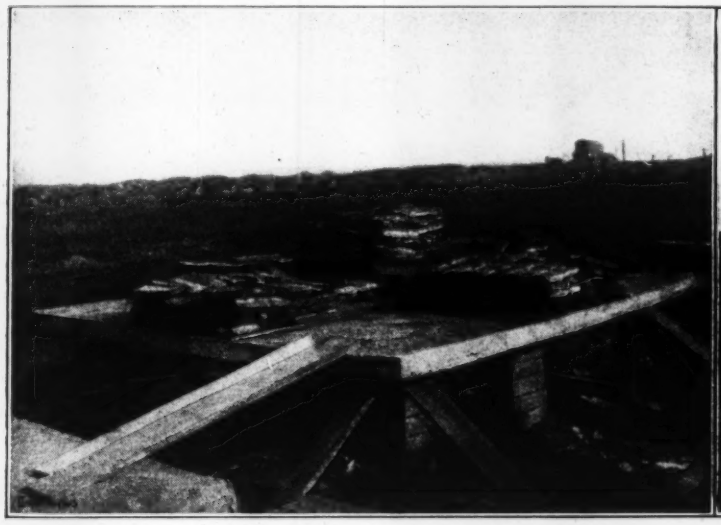


FIG. 4. UNDER LOADING

FIG. 5. FAILURE

FIGS. 4-5. NORCROSS SLAB UNDER TEST

In the Norcross slab

$A = 11 \times 0.11 = 1.21$ sq.in.;
$j = 0.9$;
$d = 5.75$ in.;
$L = 132$ in. for side belts;
$= 132\ \sqrt{2}$ in. for diagonal belts;
$i = 0.69$.

TABLE II. COMPARISON OF COMPUTED AND ACTUAL DATA ON NORCROSS SLAB

		Load Number 1	2	3	4
Side belts	Average observed elong. in 8 in.	0.00205	0.00746	0.00922	0.01133
	f_s in pounds by Eq. (1)	7690	27980	34575	42500
	W_1 computed by Eq. (4)	1465	5330	6590	
Diag. belts	Average observed elong. in 8 in.	0.00195	0.00722	0.00939	0.01229
	f_s in pounds by Eq. (1)	7313	27070	35200	45900
	W_2 computed by Eq. (4)	985	3650	4745	
$4W_1$	Comp. load in 4 side belts	5860	21320	35850	
$2W_2$	Comp. load in 2 diag. belts	1970	6700	9490	
$4W_1+2W_2$	Total comp. load on slab	7830	28020	35850	
$4W'_1+2W'_2$	Total actual load on slab	15404	31154	36572	
Side belts	W'_1 actual load by Eq. (5)	2880	5800	6690	
	D, mid deflection by Eq. (6)	0.186	0.37	0.43	
	D^1 observed mid deflection	0.128	0.41	0.52	
Diag. belts	W'_2 actual load by Eq. (5)	1940	3970	4820	
	D mid deflection by Eq. (6)	0.35	0.72	0.865	
	D^1 observed mid deflection	0.22	0.707	1.02	

From the above formulas the loadings on the side and diagonal belts which should have caused the given elongations are computed and are given in Table II. It is then assumed that the total actual loads are carried by the various belts in the same proportion as the computed loads, since there is no other way of dividing the total load between the belts. This may be stated mathematically as follows:

Let

W_1 = Computed load on a side belt;
W_2 = Computed load on a diagonal belt;
W'_1 = Actual load on a side belt;
W'_2 = Actual load on a diagonal belt.

Then

$4\ W_1 + 2\ W_2$ = total computed load on slab;
$4\ W'_1 + 2\ W'_2$ = total actual load on slab.

$$\frac{4\ W'_1 + 2\ W'_2}{4\ W_1 + 2\ W_2} = \frac{W'_1}{W_1} = \frac{W'_2}{W_2} \qquad (5)$$

W_1, W_2 and $4\ W'_1 + 2\ W'_2$ being known, W'_1 and W'_2 can be computed.

Having found the actual distribution of loading, the center deflections of the belts have been computed by simple beam theory from the formula

$$D = \frac{W' L^3}{48\ E A i j d^2} \qquad (6)$$

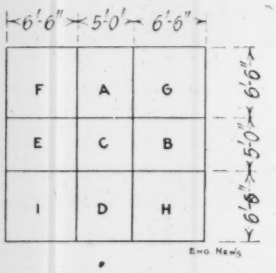


FIG. 6. ARRANGEMENT OF LOADING ON MUSHROOM SLAB

In Table II it will be noted that loading No. 1 is too small to develop sufficient elongations or deflections to overcome the initial compression in the concrete in which the reinforcement is embedded, so that the load carried by the steel is only about one-half of the actual load, the other half being carried by the concrete in which it is embedded. This is in complete accord with other similar experiments. But in case of loads No. 2 and No. 3, where the steel is stressed close to the yield point, the sum of the loads as shown by the stresses in the steel is very close to the total actual load. The stresses in the steel under loading No. 4 are so far beyond the yield point as to make computation useless.

It appears from Table II that the effect of the reinforcement is accounted for to a reasonably close approximation by considering the belts to act as a combination of simple beams, at least within the range of loading near the yield point of the steel. The steel reached its yield point under a total load on the slab of from 15 to 18 tons, and final collapse occurred under a total load of a little over twice the latter amount not distributed uniformly but piled more in the general form of a pyramid.

It was observed that the application of the relatively small loading on the corner areas *F, G, H* and *I* (Fig. 3) had a very injurious effect upon the slab, tending to break it across the tops of the columns.

The results of the test may be summarized in the Norcross system as follows:

(1) This slab is of the simple beam type, and the test shows no circumferential slab action and no cantilever action.

(2) The narrow belts running diagonally leave large areas without reinforcement, and there is consequently no provision for resisting circumferential tensions as required in slab action.

(3) The concrete showed compressive stresses on the

TABLE III. LOADS ON "MUSHROOM" SLAB IN POUNDS

Load	1		2		3		4		5	
Area	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total
A	100.8	3,276	210.6	6,552	201.6	6,552	425.3	13,822	625.3	20,322
B	100.8	3,276	201.6	6,552	201.6	6,552	425.3	13,822	625.3	20,322
C	100.8	2,532	201.6	5,040	418.3	10,458	861.8	21,546	1261.8	31,546
D	100.8	3,276	201.6	6,552	201.6	6,552	425.3	13,822	625.3	20,322
E	100.8	3,276	201.6	6,552	201.6	6,552	425.3	13,822	625.3	20,322
		Corner	Areas		Not	Loaded.				
Slab	48.2	15,624	96.4	31,248	113	36,666	237.0	76,835	348.0	112,835

Load	6		7		8		9		10	
Area	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total	Per sq.ft.	Total
A	746	23,922	772.9	25,122	899	29,222	899	29,222	899	29,222
B	736	23,922	722.9	25,122	899	29,222	899	29,222	899	29,222
C	1480.2	37,006	1569.4	39,236	1889.4	47,236	2689.4	67,326	3329.4	83,236
D	736	23,922	772.9	25,122	899	29,222	899	29,222	899	29,222
E	736	23,922	772.9	25,122	899	29,222	899	29,222	250	29,222
F			250	10,560	250	10,560	250	10,560	250	10,560
G			250	10,560	250	10,560	250	10,560	250	10,560
H			250	10,560	250	10,560	250	10,560	250	10,560
I			250	10,560	250	10,560	250	10,560	250	10,560
Slab	499	132,695	564.5	181,965	637	206,365	700	226,365	748	242,365

TABLE IV. COMPARISON OF COMPUTED AND ACTUAL DATA ON "MUSHROOM" SLAB

	Load no.	1	2	3	4*	5*	6	7
Diag. belt	Obs. elong. of middle rod in 8 in.	0.00026	0.00058	0.00071	0.00140 0.00270	0.00373 0.00610	0.00628	0.00708
	fs by Eq. (1)	975	2175	2662	5,250 10,120	14,000 22,900	23,550	26,550
	w the effective load per sq.ft.	65	145	190	350 675	933 1,526	1,569	1,800
	D comp. mid. deflec.	0.0174	0.0388	0.0508	0.0936 0.1800	0.250 0.408	0.420	0.482
	D' obs. mid. deflec.	0.0170	0.0353	0.0463	0.121 0.251	0.305 0.408	0.439	0.517
	Ave. obs. elong. in 8 in.	0.00017	0.00031	0.00039	0.00126 0.00272	0.00401 0.00607	0.00681	0.00758
Side belts	fs by Eq. (1)	637.5	1160	1460	4,725 10,200	15,000 22,762	25,500	28,400
	D comp. from w	0.0088	0.0198	0.0254	0.0467 0.0901	0.125 0.204	0.210	0.241
	D' aver. abs. deflec.	0.0100	0.0202	0.0250	0.0653 0.155	0.173 0.241	0.256	0.303

* Double readings under Loads No. 4 and 5 explained in text.

upper surface of the slab in the direction of all the reinforcing rods.

(4) The concrete showed tension at the bottom surface in the direction of all the reinforcing rods, in agreement with Norcross' own analysis.

(5) This slab deflected 1.6 in. under 33 tons and then broke down completely under 38 tons.

(6) The first crack appeared under a load of 15 tons and deflection of 0.7 in.

(7) The slab, not being reinforced on the top surface over the columns, inevitably cracks at a column when the slab is loaded around the column.

"Mushroom" Slab

The "mushroom" system is defined by C. A. P. Turner, its inventor, as a continuous flat plate of concrete, supported on columns, and reinforced in such a manner that circular and radial stresses are both provided for by metal reinforcement in the tension zone above the column, and similar provision is made for tensile stresses in the lower part of the slab concentric with the center of the panel situated diagonally between columns.

The "mushroom" test slab was 6 in. thick, and was supported on four 18x18-in. square reinforced-concrete columns distant 12 ft. c. to c. each way. These had square capitals, 42x42 in. The slab was approximately 18x18 ft. in plan, and the diameter of the outer ring rod of the "mushroom" was 66 in., while the inner ring was 42 in. These were supported on eight 1⅛-in. round radial column rods. These radial and ring rods together constitute the mushroom head proper and make a very stiff arrangement of diagonal steel in all directions, which serves to raise the regular belts of reinforcing that rest on it to the top of the slab over each column. The amount of steel so employed is, however, not greater than that necessarily used in the steel plates covering the tops of the columns in the Norcross system. The construction will be clearly understood from Fig. 3, which shows the reinforcement and forms ready for pouring the concrete. Figs. 7 and 8 show the progress of the loading and the final appearance of the slab. In this slab the belts of reinforcing rods at the bottom were lifted above the forms while the concrete was being poured by resting on cross pieces of $\frac{5}{16}$-in. round bars, except over the mushroom heads, where the reinforcement was raised until it came to view at the upper surface. The mean values of d may be taken to be: At mid side belt $d = 5\frac{1}{2}$ in. from the top; at crossing of diagonals $d = 5\frac{1}{8}$ in. from the top: at edge of cap $d = 5$ in. from the bottom.

The diagram of loaded areas for the "mushroom" slab (Fig. 6) is like that already given for the Norcross slab in every particular except that the size of the former being 18x18 ft., while the Norcross slab was 16x16 ft., the arms of the Greek cross in the "mushroom" slab are each 6 ft. 6 in. long and 5 ft. wide. Table III exhibits the loads per square foot of each of the subsidiary areas shown in the diagram as also the total loads on each of those areas.

Elongations of steel were measured by extensometers in two of the side belts and in one of the diagonal belts until the yield point of the steel was reached at load No. 8. Deflections were also measured. In Table IV these will be considered so far as they relate to the middle points of the belts. Loads Nos. 8, 9 and 10 are of great interest as exhibiting the behavior of the slab under excessive loads, showing, as they do, yielding and large permanent deformation without dangerous collapse.

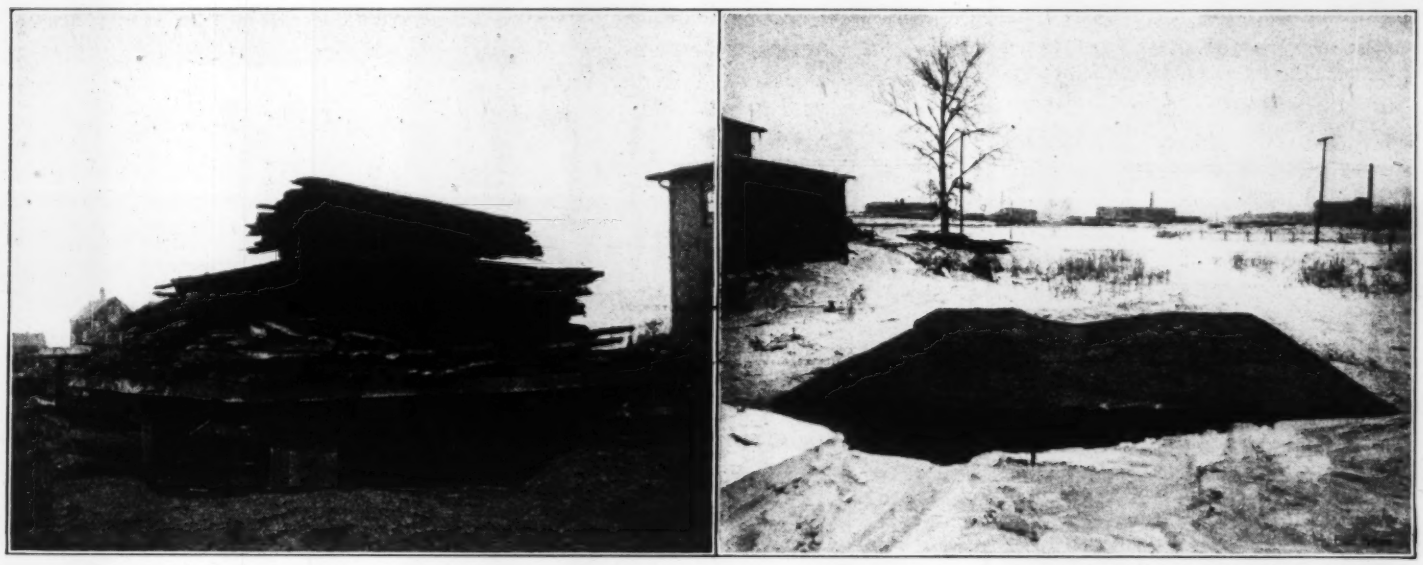


Fig. 7. Under Loading

Fig. 8. Failure

Figs. 7-8. "Mushroom" Slab under Test

The writer desires to state that Table IV is partly based on a complete rational analysis which he has himself developed for the continuous flat slab when uniformly loaded. The numerical application of that analysis is found to be in close agreement with the very numerous tests for deflection which are now available, and with the smaller number of extensometer tests of the elongation of the steel in such slabs, which have been made up to the present time. It is impracticable to develop that analysis in this paper, but it is the intention of the writer to publish the more essential parts of his slab theory at an early date. Meanwhile he will take the liberty of employing certain parts of it in computing the approximate results given in Table IV.

He has computed in the first place w, the effective load per square foot, which is the intensity of loading which according to his slab theory, would, in a continuous uniformly loaded slab of many panels cause the observed stress f_s in the steel at the center of the panel. This

computed value of w should be compared with the corresponding intensities of loading of the entire slab of the panel, and especially of the central area C. As the yield point of the steel is approached the effective load w tends to coincide with the intensity of the loading on the area C, which appears otherwise reasonable.

If now the effective load w be employed to compute the central deflection of a continuous slab of many panels carrying a uniform load of this intensity, it is found that this slab theory gives a set of deflections at the middle of the diagonal belts practically identical with those observed.

The same is true of the deflections computed for this loading at the middle points of the side belts. Slab theory shows that for this particular slab these last deflections are one-half as great as those at the center of the panel. Slab theory also shows that a continuous slab with the present reinforcement, size and thickness of the panels, etc., would show f_s at the center of the side belts to be 15% greater than at the center of the panel, as the yield point is approached. This limit is nearly reached under loads Nos. 6 and 7, and fails evidently of doing so by a small amount by reason of the lack of continuity of the slab.

The double set of values given under loads Nos. 4 and 5 is due to the fact that readings were had under load No. 4 immediately after the load was applied, and again seven days later before applying load No. 5. The second set of readings was the larger as shown. The second set of readings under load No. 5 was taken four days subsequently to the first set.

It appears from Table IV that the observed results are accounted for to a good degree of approximation up to the yield point of the steel by the slab theory.

It will be seen from Tables I and III that the first three loads were practically the same for both slabs. In the Norcross slab load No. 3, of 18 tons, stressed the steel up to the yield point, but in the "mushroom" slab the stress was so small (being in fact less than 10% of the former), as probably not to remove all the compression from the concrete in which it was embedded. Indeed the load on the latter slab became five times as much, 90 tons, before its steel approached the yield point, at which time it was carrying about twice the load which caused the complete failure of the Norcross slab.

Moreover the deflection of the Norcross slab under load No. 3 was 22 times that of the "mushroom" slab under the same load. This result is in full accord with slab theory, which shows that the central deflection of a continuous diagonal beam with fixed ends uniformly loaded with one-sixth of the total load on the slab and having the same thickness and reinforcement as the diagonal belt, would have more than six times the central deflection of the slab, while the stress in its steel would be three or four times as much. This gives a measure of the effect of slab action. By the phrase "slab action" we designate the increased strength and stiffness of the slab by reason of its resistance to circumferential stresses around the columns and around the center of the panel.

Furthermore, if this continuous beam be compared with a simple beam uniformly loaded and having the same reinforcement, the latter would have five times the deflection of the continuous beam, or 30 times that of the slab, while the stress in the steel would be one and one-half times that in the continuous beam, and six or seven times that in the slab. This last exhibits the effect of cantilever action combined with slab action.

By the phrase "cantilever action" we designate the increased strength and stiffness which is due to the continuity of the beam or slab by reason of its being fixed in direction at its supports so that it is convex upward at such points.

The concentration of the loading toward the middle of the panel such as was the case in this test will prevent any precise agreement of these numerical estimates based on uniform loading with the results of the tests, but they tend strongly to sustain our confidence in the validity of the analysis from which these concordant approximate estimates are obtained.

The amazing difference in the strength and stiffness of these two slabs, which contain practically the same amount of concrete and steel, is due to the difference of principle of their construction, which may be summarized for the "mushroom" system by considering its slab action and its cantilever action under the following counts, viz.:

(1) Circumferential slab stresses are most economically and effectively provided for by the ring rods around the column heads.

(2) The size of the "mushroom" heads is such as to make the belts so wide as to provide reinforcement over the entire area of the slab, thus securing slab action in the central part of the panel where the belts lie near the lower surface.

(3) The reinforcing belts cover a wide zone at the top of the slab over the columns and "mushroom" head, which thus provides resistance to tension, and insures effective cantilever action.

(4) Concrete is thus stressed in compression at the bottom of the slab for a wide zone around the columns.

(5) Under the breaking load of the Norcross slab, amounting to 38 tons, the "mushroom" slab deflected at first only ⅛ in., but after exposure to rain and great changes of temperature for seven days had somewhat softened the concrete, the deflection increased to ¼ in.

(6) The first crack appeared underneath the edge of the slab across the side belt under load No. 5, of 56 tons, with a center deflection of 0.4 in. and an average deflection at the middle of side belts of ¼ in.

(7) No cracks appeared on the upper side of slab at the edge, nor were any seen elsewhere, until load No. 7 of 90 tons was applied, when the yield point of the steel was evidently nearly or quite reached, giving a center deflection of ½ in.

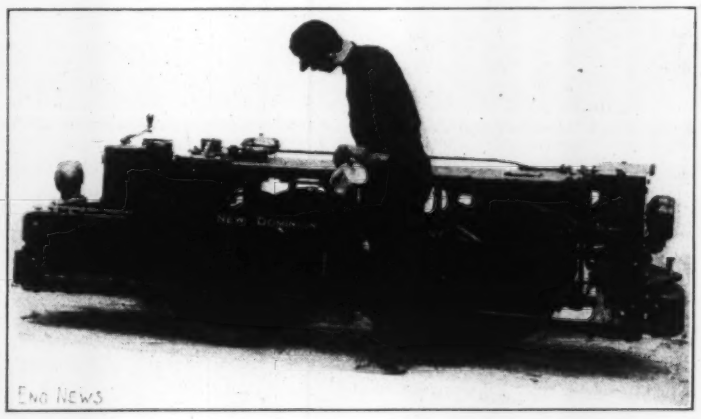
(8) The slab carried its final load of over 120 tons for 24 hr. without giving way. It demonstrated the impossibility of its sudden failure by a gradually yielding until it reached a final deflection of some 9 in., as seen in Fig. 8.

❖

The Underwriters' Laboratory, Inc., has secured 10,000 sq.ft. of additional land adjoining its present property on East Ohio St., Chicago. It plans to extend its present building to provide space for hydraulic and chemical laboratories and offices. When this addition is made the institution will have a total ground area of 26,600 sq.ft., and a total floor area of 45,000 sq.ft., devoted to various tests of materials, devices, and processes for fire prevention. It is reported that the year 1912 showed a 50% increase of volume of business handled. The completed plant represents an expenditure of $175,000, this having been contributed by the fire-insurance companies holding stock and supporting the project.

A Mine Locomotive Designed for Accessibility*

The accompanying figure shows a mine locomotive with a cast-steel open bar frame in which every attempt has been made to secure accessibility, simplicity, durability and strength. It is claimed that it will operate satisfactorily under the worst conditions with minimum attention and maintenance expense. The frame is designed to give maximum strength and to allow ready access to all parts so that the locomotive can be inspected or overhauled, when necessary, in the least possible time. The construction is the same as that used on heavy steam freight engines.



A Mine Locomotive with Cast-Steel Open Bar Frame
(Baldwin Locomotive Works and Westinghouse Electric & Manufacturing Co.)

The open frame gives much better ventilation to the motors and resistance than that obtained by plate-frame construction. The motors, brake rigging, brake shoes and sand boxes are easily accessible. The upper parts of the motors and armature bearing housings can be removed without disturbing the suspension, so that each part of the motor is exposed for inspection. To remove the grid resistance the only work necessary is to take off the locomotive covers and loosen the bolts and terminals that hold the resistance-grid frames in place.

On locomotives with outside frames, as in the accompanying figure, the Vauclain removable journal-box gib has been used. To remove a journal box with this gib, it is only necessary to drop the binder and take the weight off the journal box. The journal box may then be slipped out from the side. With frames inside the wheels, the journal-box collars are arranged to be easily dropped out for re-packing. If it is desired to take out a set of wheels and axle, this may be done without disturbing the motor suspension or connections by simply blocking the motors in place and removing the binders. The wheels may then be dropped.

The motors are a commutating-pole type, with cast-steel frames split diagonally. The axle bearings and suspension nose are on the lower half of the frame, so that the upper half, the armature, and bearing housings can be removed without disturbing the suspension or axle brackets. All windings are impregnated and the field coils are protected from vibration by heavy cushion springs placed between them and the motor frame.

This locomotive is the joint product of the Baldwin Locomotive Works, of Philadelphia, and the Westinghouse Electric & Manufacturing Co., of Pittsburgh.

*From information furnished by the Westinghouse Companies Publicity Department, East Pittsburgh, Penn.

A Comparison of Recent Bids for Various Types of Paving in the New York Navy Yard

By Walter H. Allen*

Proposals recently opened for paving in the Brooklyn Navy Yard contained bids upon several classes of pavement, both granite block and creosoted wood, and under the specifications that now govern the laying of these pavements in the Borough of Manhattan, New York City, as well as the Navy Department specifications. These bids furnish some very interesting comparisons.

The site of the Navy Yard paving is the area surrounding the recently completed dry dock. It is desired to obtain the best class of granite paving, not only on account of the heavy loads that the pavements in the vicinity of dry docks frequently sustain, but also on account of the poor success experienced in the Navy Yard with most of the pavements that have been laid there.

Bids were invited upon four classes of pavements, as follows:

Class (A) 4-in. granite cubes.
Class (B) "Durax" granite blocks.
Class (C) Oblong granite paving blocks.
Class (D) Creosoted wood paving blocks.

The specifications provide that, in the Class A pavement, a variation from the 4-in. dimension of not more than ¼ in. would be allowed, and the blocks should have parallel sides and right-angle edges. They should be laid as close together as possible in straight courses. The "Durax" granite blocks should be irregular cubes, similar to those extensively used in England, the edges to vary in dimension from 3½ in. to 2¾ in. The blocks should be laid in close contact with no joint exceeding ½ in. and in concentric interlocking segments.

Special provision was made that men employed to lay this pavement should be experienced workmen accustomed to laying this class of pavement. The oblong blocks should be from 7 in. to 9 in. in length, 3 in. to 3½ in. wide, and from 3½ in. to 4 in. in depth, depressions not in excess of ⅜ in. being allowed. Joints between courses and at ends of blocks should not exceed ½ in. This block has been extensively used in the city of Havana, Cuba. The creosoted wood blocks were to be in accordance with the usual Navy specifications and laid in the usual manner.

It was specified that the 4-in. cubes and the "Durax" blocks should show a compressive strength of not less than 30,000 lb. per sq.ft. and a coefficient of wear of not less than 15 (French standard). For the Class C paving, these figures are 25,000 lb. and 14 respectively. Bidders were required to furnish samples of blocks with the bids, and tests were to be made before the award of the contract.

The foundation for whichever class may be used will be concrete. Under the granite paving blocks, comprising the first three classes, the cushion will be fine gravel having no particles less than ⅜ in. greatest dimen-

*Civil Engineer, U. S. Navy, Brooklyn, N. Y.

sion, or chips of granite or gneiss which pass a ⅜-in. and are retained on a ⅛-in. sieve. The cushion under the granite paving is to be ½ in. thick, and the joints are to be swept full of material similar to the cushion and then filled flush with coal-tar pitch. Under the wood paving blocks a sand cushion ⅜ in. thick was specified.

The amount of grading necessary is small, averaging not far from 5 in. of excavation. Bluestone headers are required at the edges of paving adjacent to unpaved areas or pavements not upon a concrete foundation. A small part of the paving is to be done upon concrete foundations furnished by the government in connection with railroad tracks about the dry dock. On this work, only the cushion and blocks are to be furnished and placed by the contractor. Work is to begin in the spring of 1913, as soon as the frost is entirely out of the ground and the contractor is allowed three calendar months to complete his work after being notified to start.

The bids were opened on Nov. 23, 1912, and are as shown in the table below.

UNIT PRICES IN BIDS FOR PAVEMENTS, BROOKLYN, N. Y., NAVY YARD

	Class	Paving per sq.yd. including foundation	Paving per sq.yd. on foundation, laid by government	Headers, per lin.ft.
Bidder No. 1	B	4.10	3.25	1.00
Bidder No. 2	D	3.37	2.48	1.42
Bidder No. 2	D(1)	3.26	2.37	1.42
Bidder No. 3	C	5.80	4.37	1.00
Bidder No. 4	A	5.53	4.89	0.90
Bidder No. 4	C	4.05	3.40	0.90
Bidder No. 4	C(2)	3.98	3.28	0.90
Bidder No. 4	D	3.96	3.20	0.90
Bidder No. 4	D(2)	3.69	2.98	0.90
Bidder No. 5	A	7.23	5.98	1.60
Bidder No. 5	C	5.93	4.68	1.60

(1) This bid slightly modifies the government specification in regard to expansion joints and the filling of joints.
(2) New York City specification.

From these bids it is possible to draw several interesting conclusions. All bids on the 4-in. cube pavement (Class A) were very high, showing that this class of pavement, which would probably give excellent results from the standpoint of satisfactory service and low maintenance cost, is so costly to lay that except in rare instances it is impossible to consider it. Doubtless the high bids are due in part to the rather too stringent specifications regarding the variations allowed in the blocks.

The only bid for "Durax" pavement, Class B, is very close to some of the bids on oblong block pavement, Class C. This is rather remarkable in view of the fact that there is but one machine* in this country for making "Durax" blocks so that it will probably be necessary to make a large part of them by hand, and that no pavements of this class have been laid in this country and there are no experienced pavers for this class of work, so that it will be necessary to send men to Europe to acquire skill in this class of paving.

The bids for oblong granite block paving varied greatly, but it is noticeable that the low bidder also submitted an alternate bid for blocks in accordance with the specifications of the city of New York; and the difference in cost is very little, although the New York City block is larger and the specifications in regard to evenness of the surface and allowable width of joints are less rigid in the New York specifications.

*For a description of these machines and this type of paving block see "Eng. News," Mar. 28, 1912, p. 614.

Three bids were received for creosoted wood paving blocks, two in accordance with the government specifications and one in accordance with the New York City specifications. The lowest bid is in accordance with the government specifications.

The reductions in the cost of paving for the work upon the government foundation in the vicinity of the railroad tracks vary from 64c. to $1.43. These figures do not fully represent the estimated cost of foundation alone, but probably involve the different opinions of the contractors as to the amount of excavating to be done and the difficulties of paving about the railroad tracks.

Undoubtedly, some of the contractors were influenced in their bids by the rigidity of the government specifications and their unfamiliarity with work for the Navy Department, some bids being out of all proportion to the actual cost of the work.

The most interesting feature of the bids is that the one for the "Durax" pavement is among the lowest of all the bids for granite paving, although, due to the novelty of this pavement in this country, it was to be expected that the bids for this class would be much higher than those for the oblong blocks. It indicates that with some familiarity and experience in laying this pavement and with better means for getting out the blocks, "Durax" pavements can be constructed in this country at a less cost per sq.yd. than the best class of oblong granite blocks which are found in the streets of the large cities of the United States.

Graduate Industrial Courses for Electrical Engineers

A development of the older courses for student engineers in the testing department of the General Electric Co. is being made by giving them experience for a period not exceeding three months under the consulting-engineering department. Student engineers in good standing in the testing department for not less than one year are eligible. The work will consist in assisting one of the consulting engineers, and the time spent will depend on the time required to complete the particular work in a project, and may be from a few days to several months. A student engineer may thereafter return to the testing department or assist in some further work under the consulting-engineering department, depending on conditions in the departments at the time.

Furthermore, a general extension course under the direction of the consulting-engineering department has been made available to a limited number of student engineers who must have been in the testing department not less than a year, must have had some previous experience under the consulting-engineering department, and must have been recommended either by that department or by the transformer department.

The course extends over seven months. Of these at least four are spent in the consulting-engineering department, and three in the transformer-engineering department, or other allied divisions. While under the former department, the student engineer is given work with various classes of theoretical, practical, experimental, and development work, being assigned to the different consulting engineers. On each Saturday forenoon weekly lectures are given by prominent men.

EDITORIALS

Excess Water in Spouted Concrete

Among the many devices to reduce the cost of transporting concrete from the mixer to its place of deposition, no one has had a more successful and rapid growth than the spouting or gravity chute. If a convenient central station can be located, this system provides the advantageous combination of a permanently placed mixing plant with flexible conveying system, plus the marked economy of "letting gravity do your work," as one of the spouting systems advertises. It is small wonder that such superior merits have resulted in wide use.

Unfortunately, however, we live in an imperfect world, and these admirable contrivances partake of this imperfection. Unless properly designed and operated, they are very apt to produce a water-logged concrete, or, if that term is not precisely appropriate, an excess of water in the concrete deposited. Some months ago a correspondent in these columns called attention to this possibility and suggested the urgent need of tests to determine the effect of excess water in a concrete mixture. The article by J. F. Greene on the Latah Creek Bridge at Spokane, on another page of this issue, emphasizes this need by reference to a specific case.

Mr. Greene's experience is a common one. Concrete, when mixed to what has been generally supposed to be a proper consistency, has a fairly high friction coefficient, and to get it to flow easily one of three things is necessary. Either the grade of the pipe or trough must be high, auxiliary human pushers must be employed or a large proportion of water must be used in the mixing. The first of these alternatives is a matter of original design in a gravity plant and a proper elevation is often an impossibility. The second adds just the extra cost the system is designed to avoid. There remains then only the third alternative, which is the one almost invariably resorted to. The consequence is that in many installations of this type of conveyor, concrete is sluiced down the trough or pipe in an entirely too liquid condition, with the results as noted by Mr. Greene.

The difficulty lies in determining at just what point super-saturation commences. The old quibble over wet and dry concrete is a thing of the past. No one now doubts the efficacy of a certain degree of fluidity in concrete. On the other hand, no one can doubt that too much water is a dangerous thing—for concrete. As the matter now stands, there is no definite dividing point between just enough and too much, and the opportunity for the testing engineer is great.

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Another Flat Slab Test

The comparative tests of two different types of flat-slab floor, described by Dr. H. T. Eddy on another page of this issue, are very illuminating in spite of some very serious defects in the plan of the tests and a certain incompleteness in the analysis of the stresses. As an indication of the initial strength of a well designed flat slab and of that no less desirable quality, the ability to fail slowly when failure is unavoidable, the tests could not be better. But the construction of the so called Norcross slab and the methods of analyzing both slabs leave much to be desired.

In the first place no one should ever think of building in an actual structure a flat-slab floor of the type called the "Norcross slab" in the tests. This panel may approximate in design the specifications of the Norcross patent; but the lack of any negative moment reinforcing at the columns, the extreme narrowness of the diagonal belts and the absence of any spreading bars to provide cantilever action over the columns all mark the slab as impracticable. The test results only confirm the impression that the design suggests. It is probable, however, that the slab would have shown up much stronger if the same amount of steel in the diagonal bands had been widened so as not to leave any unreinforced concrete area, a very simple amendment of the system. The transmission of the loads through so great an area of plain concrete to the concentrated steel undoubtedly subjected the concrete to undue stress and caused cracks (not noted in the test) which developed to such size as to cause the observed sudden collapse of the slab.

In regard to the analysis of this slab, Dr. Eddy's figures certainly show that the simple beam theory can account for stresses and deflections in this type of construction with remarkable fidelity, in spite of the obvious fact that a series of beams integrally tied together can not act in the same fashion as the same beams separated into units.

The analysis of the "mushroom" slab is equally interesting, although as yet so incomplete as to be of no value to those numerous engineers who are seeking a rational method of analysis of this highly successful type of construction. It is to be hoped that Dr. Eddy will soon make public his promised method of analysis and thus show conclusively the accordance between the tests here discussed and a rational theory of stress. Meanwhile, publication of the tests is full and open so that any engineer may make his own deductions therefrom.

⁂

A British Defense of the Open Fireplace

A long-standing puzzle to Americans resident in England is the tenacious hold of the open fireplace as a method of heating dwellings. While steam and hot-water systems have long been in general use for heating public buildings, it is extremely rare to find in England any other means of heating in a private residence, even the most expensive, than the open fireplace.

An American accustomed at home to normal living-room temperatures of 70° maintained throughout the cold-weather months as a matter of course, finds himself, on taking up residence in England in the winter, in constant discomfort from the cold. In severe weather, the

open fireplace, which is usually very tiny in proportion to the size of the room, can only make comfortable persons who are close beside it. Remote parts of the room are exceedingly chilly and halls and passages are wholly without heat.

The American stranded in such a chilly place will recall longingly the hot-air furnaces and steam radiators of his native land; but let him not hope to make a fortune by educating Englishmen to the comfort attainable by such simple means. The Englishman accustomed all his lifelong to the modicum of heat that the hearth fire affords, prefers that mode of living to the artificial summer temperature which the American regards as a primal necessity.

A curious correspondence has been recently going on in the columns of the London *Times* Engineering Supplement respecting the use of anthracite stoves for heating living rooms. This correspondence draws forth a vigorous reply in the *Times* of Feb. 19 from a correspondent who rushes to the defence of the open fireplace as if it were the British constitution itself that was under attack. We quote from his letter as follows:

> An open fire has great charms in its romantic play of flame and ever-changing form; it varies the monotony of life and leads us to exercise ourselves in the making of it up. We are disappointed at its dullness or charmed with its brightness, and we regard it almost as an individual, one of the household, and gain from it something of the stimulation of and variation in our mental life which we gain from intercourse with a friend. It has indeed been the best friend of man through vast ages of his earthly pilgrimage, and is interwoven with the poetry and romance of life.
>
> The anthracite stove is warm but an ugly clod of a thing, and cannot touch the spirit of man. If the open fire were always made to fulfill the conditions of complete combustion, and did not darken the atmosphere, we would not give it up for any of the new-fangled inventions of the heating engineer. Is not the extra money spent on the coal well spent if it contributes to our health and happiness? Is not the open fire as valuable to a cultured Englishman as his pictures and his books?

The Greatest Piece of Engineering Work in the World

Measured by the amount of money to be expended, the largest piece of engineering work ever undertaken in the world, not even excepting the Panama Canal, is the New York Rapid Transit Subway System, on which the final steps were taken last week by the signing of the final contracts between the city and the Interborough and Brooklyn Rapid Transit Companies. According to the official statement made by Borough President McAneny at the meeting of the Board of Estimate on Mar. 18, the total expenditure on the extensions of the Interborough System, for both construction and equipment, will amount in round numbers to $140,600,000, and on the Brooklyn Rapid Transit System to $161,200,000. This makes a total of nearly $302,000,000. Adding to this the expenditure on third-tracking and other improvements on the elevated lines and such work as the Steinway tunnel, which is to be included in the new system, the total investment will approximate some $330,000,000. Adding to this the investment already made in the existing subway system, we arrive at a valuation of the entire subway system when completed which will considerably exceed the cost of the Panama Canal work.

As our readers are aware, a considerable mileage of the new subway work is already under contract. The total amount of contracts already let is some $75,000,000. Contracts are still to be let on the work to the amount of $160,000,000.

The amount of engineering work involved in the construction will be considerably greater than that involved in executing the comparatively simple work on the Panama Canal construction. There are now about 750 employees on the engineering staff of the Public Service Commission and 200 to 250 more will be necessary. The most important and most difficult part of the engineering work was the preparation of the general design, which has already been completed. What remains to be done is chiefly supervision and inspection of the actual construction work.

The importance of the new lines in adding to the transit facilities of the great city can hardly be overestimated. As to whether their earnings will be sufficient to pay interest on the enormous investment involved, there are so many varying and conflicting opinions that we shall venture to express none here. This much, however, is certain, that the city's representatives in the Public Service Commission and the Board of Estimate, who have given years of study to the subject of transit development and to the best methods of securing it for the city, are satisfied that the great development now to be undertaken is necessary for the city's welfare, and that the terms on which it is to be carried out represent the best that it was possible for the city to secure.

According to the contracts just signed, the entire system is to be completed and ready for operation by Jan. 1. 1917.

Stereophotogrammetry

In view of the tremendous progress in every branch of engineering practice during the past century, it is remarkable that so little progress has been made in the work of surveying, upon which practically all other civil engineering work is based. We still have to actually measure the ground and afterward reproduce those measurements, reduced in scale, upon paper. It is true that there have been great improvements in surveying instruments, resulting in much greater accuracy; but the essential elements in the theory and practice of surveying are the same today that they were when George Washington surveyed in the forests of Virginia, using a compass for angle measurements and pacing the distances.

The introduction of photography as a means of obtaining information for topographical maps has been practically the only radical change in survey methods in a century; yet photogrammetry is a term none too familiar to American engineers and surveyors, considering the wide possibilities of the art. As a method of surveying, its use in this country, we believe, has been limited to a few government surveys, and these have been made principally in Alaska. Regarding this work in Alaska we are informed by J. W. Bagley, topographic engineer of the United States Geological Survey, as follows:

> A method of photo-topographic surveying for mountainous country has been used with success by the Geological Survey in Alaska since 1910. A panoramic camera designed with considerable precision has taken the place of the phototheodolite or plate camera. It is used in connection with the planetable to supply data for constructing the contours. The most evident advantages in the field work are the substitution of films for plates, thereby reducing the weight of equipment,

the lessening of time required to take three views instead of eight, and the greater convenience of preserving and transporting exposed films rather than plates.

In the compilation of data the economy is even greater. An apparatus designed for the rapid transference of horizontal angles to the map places the photograph in the same position relative to the map as that portion of the landscape lay when the sheet was properly oriented at the field station. This relation can be easily established at any time, so that sights may be taken very much as sights are taken on the planetable by means of an alidade, without the necessity of noting readings. Elevations can be computed from the photographs at a rate somewhat greater than from telescopic angular readings.

The method has proved sufficiently accurate to give excellent results on a scale of a mile to the inch, using 50-ft. contours. The photographs can be relied upon to furnish elevations at distances up to five miles within 20 ft. The cost of the work has been less than the cost of work by the familiar planetable method.

It is doubtful if any American engineering college gives actual practice in photo-surveying, although the principles of photogrammetry are now included in some standard textbooks on surveying.

In Europe, on the other hand, the practice of photogrammetry has steadily developed, and this development has now reached a high degree of perfection, evolving a new term "stereophotogrammetry," quite unknown to the vast majority of American engineers.

Stereophotogrammetry, moreover, is not so formidable an art as its name implies. Whoever is familiar with an ordinary picture stereoscope can quickly appreciate the possibility of taking measurements of points on the pictures in all three dimensions. The perception of depth in stereoscopic pictures is the result of looking at two photographs taken simultaneously on two plates which were a distance apart of only a few inches. Imagine then how much the perception of stereoscopic depth may be increased if the plates are exposed at a distance apart of 50 or 100 ft. Place these plates in an apparatus designed to still further increase the stereoscopic effect and to magnify the image—increasing the stereoscopic effect a thousand times over that of an ordinary picture stereoscope—and we are able to understand how it is possible to make measurements by stereophotogrammetry with no mean degree of accuracy.

The most serious objection to stereoscopic measurements is due to the large personal equation involved. The ability to see stereoscopically varies much with different individuals; some persons are unable to perceive stereoscopic depths at all. Such a personal element is, of course, a drawback to any method of making measurements. Stereophotogrammetry has, however, made such progress in Europe that American engineers cannot long neglect giving it a trial, and we believe that the article elsewhere in this issue describing the progress and methods of photo-surveying in Europe will be read with interest by all American engineers interested in survey methods.

Stereophotogrammetry has wide fields of usefulness in many branches of art and science. Its application to architecture can readily be appreciated. By means of two photographs taken at the ends of a short base line, it is possible to take all the dimensions of an inaccessible dome or tower. It has been used to study the movement of glaciers; photographs taken from the same stations at intervals of time make it possible to perceive very slight movements in the ice. Stereophotogrammetry has also been used to study the height and length of sea waves; to study and measure the features of the human face as an aid to sculptors; to study the stars, and for a variety of other scientific and useful purposes.

Stereophotogrammetry so far extends and simplifies the practice of photogrammetry that it is perhaps the most interesting and revolutionary development which has ever entered into the practice of surveying. Think of the possibility of sending a hair-line marker from point to point of the virtual landscape and taking all the measurements of those various points with many times the ease and speed with which a stadia rodman could cover the distances in the actual landscape! It seems certain that this method will be extensively applied to obtain topography of large areas throughout the world, of which no topographical maps now exist. For small-scale topographical details, not requiring a high degree of accuracy, stereophotogrammetry seems to offer great advantages over any of the old time methods.

How much effect these recent improvements in photo-surveying will have on railway-location practice remains to be determined. By means of stereophotogrammetry it will be possible for a chief engineer to examine every mile of a projected railway line without stirring from his office door. It will be possible to send a two-man party into the ruggedest country and obtain in a few weeks all the necessary data for a preliminary location. In mountainous countries where no previous maps exist, it cannot fail to be of great advantage, for by use of the stereophotographs it will be possible to readily and quickly make measurements which could not be made in the field without elaborate triangulation.

Proposed Water-Supply and Sewage Disposal Legislation in Minnesota

Minnesota has been so progressive in some public health matters that it seems strange that it has so long delayed taking the necessary steps for adequate sanitary control by the state of water-supply and sewage disposal. A bill is now before the legislature which, if enacted, will do much to place Minnesota in line with Massachusetts, New York, New Jersey, Pennsylvania and Ohio, in this respect. The bill might well be strengthened at some points and made more specific in others, but perhaps it goes as far as can be expected at the beginning of such legislation.

In substance, the bill provides that all persons, corporations and municipalities other than first-class cities supplying "water for private or public consumption" or disposing of sewage within the state must, within six months of the passage of the bill, file with the State Board of Health certified copies of plans of their water or sewerage systems, with descriptions of the sources of water-supply and places of sewage disposal; and that "no *other* (italics ours) source of water-supply or place of sewage disposal shall be adopted until complete plans therefor have been submitted to the State Board of Health and a favorable report shall have been made thereon by said Board." The provision that no other source of water-supply shall be adopted without a favorable report thereon by the Board is supplemented by a section providing that no water-works shall hereafter be built to supply "water for private or public consumption" until the plans therefor have been submitted to the Board and favorably reported on it by it.

As the bill contains no other provisions regarding

plans for water-supplies, sewerage or sewage disposal it will be seen that it does not go so far and is not so specific in giving the Board control of these matters as is common in the legislation of other states. No one would be required to file plans of sewerage systems built later than within six months of the passage of the bill, and no control over plans for sewerage systems is given the Board, except as regards new *places* of sewage disposal. One would suppose that if it were essential to have plans of existing sewerage systems filed with the Board it would be equally essential to have plans for new systems filed, or at least plans for sewage disposal *works*. However, judiciously exercised state control over the selection of sources of public water-supply and places of sewage disposal would be a long step in advance for Minnesota and for many other states which have thus far let this important sanitary and health matter go by default. It is well to note that the pioneer in state control of water-supply and sewage disposal, Massachusetts, has not extended the jurisdiction of its State Board of Health beyond *sources* of water-supply and *modes* (not places) of sewage disposal; and that the action of the Massachusetts board stops short with "advice"—but in practice this advice governs.

State approval of sources of water-supply, of plans for water-works systems, and of places of sewage disposal, valuable as such approval may be, is only a start in the right direction. Unless public opinion in support of a state board is remarkably strong, means should be provided to insure that plans as approved are executed and then that the works built in accordance with those plans are not only properly operated but are modified in design, construction and operation as may be necessary to meet the original or changed conditions.

The Minnesota bill contains a section on the operation of water and sewage purification works which is good as far as it goes but which is weak because it makes no provision for such water-supply and sewerage systems as do not treat the water or sewage, and also because the Board can do nothing but make "recommendations" for changes which no one is bound to follow. We quote as follows:

> Sec. 6. Whenever the State Board of Health finds upon investigation that any water or sewage purification works, becaust of incompetent supervision or inefficient operation, are not producing an effluent as pure as might reasonably be obtained from such plant, and by reason of which any water-supply has become dangerous to health, or any stream or body of water has become offensively polluted or has become a public nuisance, said board shall issue a notice to the proper officer, board or department of any municipal corporation, to the proper officer of a private corporation or company or to the person or persons having charge of or owning such a plant, apprising him or them of the conditions found, together with recommendationss as to the proper procedure necessary to remove the danger.

It is obvious, as already stated, that this section does not go far enough. Possibly existing legislation supplements this section, but we think not. To fully control, the State Board of Health should have power to require cities or companies which are supplying impure water to install purification works and should also be enabled to compel the abatement of sewage disposal nuisances.

The penalty clauses of the Minnesota bill appear to apply only in case of failure to comply with the requirements to file plans, as already outlined. To make the bill fully effective the penalties should be extended to cover failure to follow the plans filed with the Board and failure to follow advice as to needed changes in water purification and sewage treatment works. Moreover, the money penalty (a $50 to $100 fine, regardless of the period of non-compliance), appears ridiculously small, especially as the alternative of 60 to 90 days imprisonment in a county jail seems to stand little chance of ever being applied.

We have reviewed the provisions of the Minnesota bill because legislation along this general line is greatly needed in other states than Minnesota and because upon our engineering readers rests in large measure the duty of seing that such legislation, wisely framed, is enacted with as little further delay as possible. If the engineers of Minnesota, both individually and as members of the local and state engineering societies, will rise to the occasion, they can doubtless insure not only the enactment but also the strengthening of this bill—which gives evidence of having been drawn in moderate terms lest a stronger bill should be defeated. Certainly the engineering and law libraries of the twin cities must contain the statutes by which other states have vested control over water-supplies and sewage disposal in their state boards or departments of health. These statutes should be studied—but in the light of their actual workings in daily practice.

LETTERS TO THE EDITOR

A Bill to License Engineers in Iowa

Sir—I inclose a circular letter which is being sent out to engineers in Iowa by a special committee of the Iowa Engineering Society, urging them to use their influence with legislators to secure the passage of a bill now pending in the Iowa Legislature to license engineers.

Especially noteworthy is the argument which the committee uses in support of the bill. I quote as follows:

> Anyone may practice without license, if he wishes, and anyone now in practice may have a license without examination.

* * * * *

> The proposed law is a protection to the public similar to that now afforded by laws regulating the practice of Attorneys, Physicians, Pharmacists, Dentists, Veterinarians, Undertakers and Nurses.

If anybody now in practice may have a license and anybody else may practice without a license, the puzzle is to find where the "protection to the public" comes in!

The special committee further declares that "the engineers and surveyors are willing to pay all the cost." How does it know they are willing? Perhaps some may fail to see where they will receive any benefit to compensate for the tax.

The special committee voluntarily classes engineers

with veterinarians, undertakers and nurses; but I doubt if the profession as a whole will consider the movement a well directed effort for the uplift of the engineering profession.

N. A. Y.

Mar. 19, 1913.

[The circular letter inclosed is as follows:—Ed.]

Dear Sir: The bill for licensing Civil Engineers and Surveyors has been introduced in the Senate. It is known as Senate File No. 434.

The Committee has exercised its best efforts in getting this bill before the General Assembly and the question as to whether it will become a law must now be answered by each man in the practice for himself as its passage will depend on individual action.

So far as we know, there is no opposition to this bill. But the inertia of the Legislature, combined with the inevitable rush at the close of the season, will defeat it unless individual influence is brought to bear on the legislators by those of us who are interested.

You are urgently requested to write to your senator and representative, asking for the passage of this bill, if you believe that it is a step in the right direction. As the time is short, the letters must be written at once in order to be effective.

Here are some of the points of the bill which you may advantageously mention in writing to your legislators:

The proposed commission is self-supporting and will require no appropriation of public funds.

Anyone may practice without license, if he wishes, and anyone now in practice may have a license without examination.

The proposed law is a protection to the public similar to that now afforded by laws regulating the practice of Attorneys, Physicians, Pharmacists, Dentists, Veterinarians, Undertakers and Nurses; AND THE ENGINEERS AND SURVEYORS ARE WILLING TO PAY ALL THE COST.

J. L. PARSONS, THEO. S. DeLAY, SETH DEAN, Special Committee, Iowa Eng. Society.

Experience with Wood Stave Pipe

Sir—With reference to the article on wood-stave pipe for irrigation, by R. K. Tiffany, in your issue of Feb. 6, 1913, I beg to give my experience to corroborate the conclusions reached by Mr. Tiffany.

The Burbank Co. took over the uncompleted Burbank irrigation project in September, 1912, and immediately commenced the rehabilitation and improvement necessary to place the property in suitable shape for operation. The project consists of 14,000 acres of very arid sagebrush land in the western end of Walla Walla County, Wash., in which the soil and climatic conditions are almost identical with those described by Mr. Tiffany.

Our experience with one pipe line has been especially illuminating. It was an inverted siphon 6200 ft. long, of 38-in. continuous wood-stave construction, which was built five years ago and had been used for only four irrigation seasons. It had been buried originally, but the wind had blown away the light soil so as to leave the top of the pipe partly uncovered. One section of the pipe in the bottom of the depression was partly submerged and covered with damp soil for a distance of about 300 ft. During the last irrigation season, it became so badly impaired as to be unreliable for further service. Upon investigation, it was found that the staves had rotted from the outside only; but where the pipe had been uncovered, and where it had been submerged and covered with damp soil, the staves were absolutely sound. This information indicated three general conclusions: (1) that wood-stave pipe, without a preservative coating, buried in dry volcanic ash soil and not under hydraulic pressure continuously, would be subject to rapid decay; (2) that its life would be much greater if the staves were exposed directly to the air, and (3) that rapid decay would not take place if buried in wet volcanic ash soil.

In reconstructing this pipe line, it was concluded that it would be best from every standpoint to entirely rebuild the line, using new staves and tongues, but employing the old bands and shoes, placing the pipe line entirely above the ground. To accomplish this purpose, cradles or mud sills were placed under the pipe 6 ft. apart. The cradles were made of 6x8-in. common fir lumber, 4 ft. long, set on edge. The tops of mud sills were milled out to a maximum depth of 3 in., to a curve of 20-in. radius [giving a curved bearing with a chord of about 20 in.—Ed.].

These cradles were sawed in 4-ft. lengths and milled at the sawmill, the additional cost for milling being $5 per M b.m., or approximately 8c. additional for each cradle.

Such construction will make the maintenance of the pipe line much simpler, as all leaks can be immediately detected at their inception, and repaired, and thus make it possible to keep the pipe line full of water during the winter season, except in freezing weather.

The pipe line was reconstructed along these lines and we believe that under the circumstances a reasonably long life will obtain. The Burbank Co. has constructed several miles of continuous wood-stave pipe in the last six months, and this construction has been followed throughout.

In another instance on the Burbank project, an inverted siphon 1200 ft. long and made of 14-in. machine-banded pipe, heavily coated with tar, had been constructed five years ago and buried under 2 ft. of earth. It was found to be in exceptionally good condition. We credit this result to the heavy protective coating and confirm the conclusions reached by Mr. Tiffany.

The Burbank Co. is installing several miles of machine-banded pipe for distribution purposes, and in doing so, departs from its practice with the larger size continuous staves pipes, by first providing a heavy coating of asphalt, about $\frac{1}{8}$ in. thick, and then burying the pipe fully 18 in. below the ground surface. The object is to place the pipe lines below the frost line so that they may be kept full of water, not only during the irrigation season, but during all other times as well, thus preventing access of air to the interior of the pipes and relying upon the preservative coating to prevent decay from the exterior.

It is apparent, therefore, that the general results of our observations coincide identically with those of Mr. Tiffany and we differ only in the methods employed in overcoming the difficulties for the continuous stave construction.

ELBERT M. CHANDLER.

Burbank, Wash., Feb. 13, 1913.

Sir—The article by R. R. Tiffany in your issue of Feb. 6, 1913, giving experiences with wood-stave pipe leads me to contribute notes on a similar experience.

In 1903 a 14-in. wire-wound stave pipe was laid by the city of Tacoma along tide-land streets which had been filled by dredging sand from the adjacent waterways. The pipe was constructed of $1\frac{1}{8}$-in. fir staves, with $\frac{5}{16}$-in. bands spaced in winding approximately $1\frac{1}{8}$

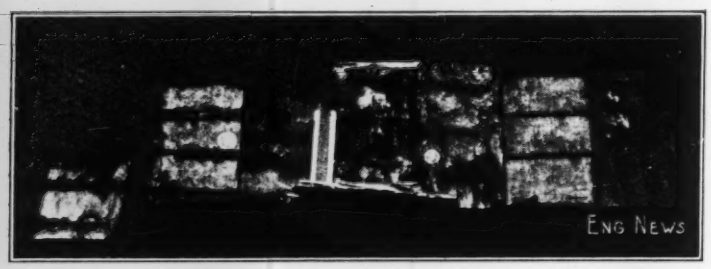
in. c. to c. and was under a pressure of 106 lb. per sq.in. This pipe was in a sawmill district and connected with a large number of hydrants which were used a great deal for sprinkling and frequently for fire service.

Frequent repairs were necessary on account of leakage through open seams and usually cut bands were found. The open seams were attributed to the careless use of hydrants, causing water hammer, thereby increasing the already high pressure. The consequent leakage churned up the sand backfill, cutting the bands and so aggravating the trouble. This pipe was replaced in 1910 with a cast-iron main.

During the summer of 1912 a section of 31-in. stave pipe, laid in 1899, was replaced. The pipe was uncoated. It was laid in a light, sandy loam and covered two feet above the top with the same character of soil. At this point the pressure was 40 lb. per sq.in. Fir staves, 1⅜-in. thick were used with coated iron bands and shoes.

The staves were found to be water-soaked and in fair condition on the inside. On the outside, the bottom staves were good but a small amount of rot showed along the sides above the spring line of the pipe, with excessive rot along the top staves.

The accompanying view shows sections of staves from



WOOD STAVE PIPE AFTER USE AT TACOMA, WASH.

this pipe. The two right-hand sections show the condition of the inside and outside of staves at the bottom of the pipe. The other sections are the outside of staves from the upper part of the pipe. The coins are 25 and 50c. pieces and the scale is 6 in. long. The metal tongues are shown on two sections.

The right-hand section shows approximately $\frac{1}{16}$ in. of soft wood on the inside, a portion of which has been scraped away to sound wood. This surface appeared to be in excellent condition when first taken out, but it rapidly dried and checked. This was the general condition on the inside of pipe, at all points.

Along the top of pipe there was not over one-half inch of sound wood, that part between bands not having sufficient cohesion to stay on the staves while being removed. The coins are resting on the ridge of wood under the bands.

In replacing this pipe most of the original coated bands were used again and all of the malleable-iron shoes.

Another section of this pipe, where uncovered a few blocks away in a clay soil and five feet back of backfill, was in perfect condition on the outside.

Corroborating the experience of Mr. Tiffany, the drying out of our backfill during the summer heat with the absorption of the surface moisture from the staves was the cause of our trouble.

Mr. Arthur L. Adams in Vol. LVIII, Trans. Am. Soc. C. E., states several of the important points of his experience on the Astoria Water-Works, three of which agree with general conditions here on uncoated pipe as follows:

> Where buried, its durability has depended upon the soil conditions and the depth of backfill.
>
> When the depth of backfill has exceeded two feet above the pipe, and the material has been free from vegetable matter, and has been of a fine and impervious character, much less deterioration has taken place.
>
> Wherever the staves have been in contact with loamy earth or earth containing vegetable matter, or wherever they have been covered with porous material, or to a depth of less than 2 ft., rapid decay has resulted.

The replacing of the above noted pipes came under the direction of W. B. Short of the Tacoma Water Department who has the sections shown in his museum.

F. H. CUSHMAN,
Assistant Engineer.

Tacoma, Wash., Mar. 3, 1913.

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The Status of the Engineer

Sir—I was surprised to read the arguments of Mr. C. S. Burnett in ENGINEERING NEWS of Mar. 13. Does not Mr. Burnett know that such arguments as "let the employer judge," "the better man will climb to the top of the ladder," etc., are old and stale? They were and are used against union laborers all over the world. Yet the facts show that wherever a union of laborers does not exist the wages are brought down by the employer to the starvation rate.

How many of the so called experts do we find at the top of the ladder of a municipal staff of a big city like New York, Philadelphia and Chicago? How many experts do we find in the big bridge or railroad companies which employ hundreds of engineers? We find very few, and most of those few climbed to the top of the ladder through political or some other backing, while the rest of the hundreds are toiling in the offices or fields like drudges, and very often for salaries less than the organized laborer gets.

I suggest that engineers should delve a little not only into the engineering sciences, but also into political economy, history, sociology and modern philosophy; then they will be able to work out their salvation.

S. RESWICK.

Barge Canal, N. Y. State, Mar. 17, 1913.

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Legislation to Restrict the Practice of Engineering

Sir—The writer would like to add his mite to the present discussion on "The Status of the Engineer."

Analogies drawn between engineering and the older professions, such as medicine, law and theology, fail because of one critical point not emphasized by those who have heretofore discussed the question.

In all these older professions no false hopes are held out, to the youth who proposes to enter them, of large incomes and ultimate affluence. Statistics have been published and novels have been written to acquaint people with the fact that the young lawyer, or physician, or clergyman, has an exceedingly small income, on the average, and a dismal old age to look forward to unless Dame Fortune is good to him.

These young fellows, having taken up their chosen professions with no false hopes, realize that only a very small proportion gain more than a fair living and that

very many never make a living, being dependent upon friends and relatives for many years.

In 1907, Mr. Onward Bates in his Presidential Address before the American Society of Civil Engineers, presented a study of occupations of the active members of the society and showed that more than three-fourths of them were dependent upon salaries for compensation and less than one-fourth received fees as compensation. He neglected to take into consideration that every member of the society, in whatever grade, styles himself an "engineer" and therefore the study should have included Associate Members and Junior Members. The percentages would have been decidedly altered by such a study. Less than 10% of the men who style themselves engineers are in private practice.

On the other hand, less than 10% of the young fellows who enter engineering schools understand that the engineer is a "hired man," connected throughout his life with seasonal labor. A few secure positions where they may become permanent fixtures. A few meet with success. The percentage is perhaps about that of every business in the world.

There is much to be regretted in the fact that conditions of pay and employment in engineering work are not better; but this is not the most serious trouble. That lies in the fact that the students entering engineering schools do not know as much about the possibilities of engineering and the probability of success, as do the freshmen in the law or medical or theological schools about the professions they have chosen. Highly colored advertisements of certain schools and the standing advertisements of numerous corporations employing large numbers of very low-paid draftsmen, deceive parents and youths alike. Even many school teachers are deceived. The general impression in all quarters is that engineering is a remarkably high-paid vocation.

Let the truth be known, and discussions of ethics will stop. Men who have been led to take up the study of engineering through a misunderstanding of the work will avoid it and the men who do go into the work will have a sincere liking for it and have less of the commercial spirit, which is really what interferes with a decent attitude of one competing engineer towards another.

The writer is opposed to legislation restricting engineering practice. He has always been opposed to it; but at the present time he finds himself in the position of advocating the passage of a license law for structural engineers in the state of Illinois.

In 1897 the architects of the state had a license law passed to make a profession of architecture. All architects then practicing in the state who had been in practice for three years preceding the date of the passage of the act, were given licenses without examination. A few structural engineers of standing were also granted licenses in the same manner. The law was passed ostensibly to protect architects in their practice against ignorant contractors and builders and provide the public with safe buildings, from a structural and sanitary standpoint.

At first the examination papers were so marked that a number of engineers were able to pass; but lately, while the character of the examinations given has not altered, (and in all candor we must admit they are very fair), the methods of marking and the weights given to each subject have changed so that it is well nigh impossible for engineers not accustomed to *artistic* architectural drafting, to pass the examination. Thus the act has now become a law to protect the architects against the competition of engineers; and the work of the Board of Examining Architects, and the decisions of the legal advisers of the board, are against engineers; all this despite the fact that a modern building is more of an engineering than an architectural proposition.

The writer is one who has felt severely the effects of the law and with others he has been attempting to come to some understanding with the architects so that owners desiring to erect certain classes of structures may exercise some option as to the employment of either an architect or a structural engineer for proposed new buildings. The larger corporations employ in their office licensed architects merely to comply with the law requiring the seal and signature of a licensed architect Some of these salaried architects are competent men, but some get low salaries and are employed merely as draftsmen, their ability as designers of the classes of structures their employers erect being markedly lacking.

So in Illinois we who must live are working to secure some legal rights for engineers as against architects; and a number who are on principle, opposed to any legal protection for engineers, find ourselves confronted by facts, not theories, which make legal protection become necessary.

The matter was first agitated six years ago; but the architects are very clever men and nothing was done by the committees appointed by engineers. Four years ago the same thing happened and again two years ago the clever architects managed to switch the committee of engineers on to the preparation of an impossible dream, the codification of the building laws of the state. Such a measure would be fought pretty generally for it adds to the cost of building; and a building ordinance suitable for Chicago cannot be enforced in a smaller place. This year the Legislature is once more in session and the engineers are talking and not doing.

The architects act together. Of the men who call themselves engineers, 90% are on salary, yet they persist in taking prominent part in discussions and some high-salaried corporation engineers are usually selected to head committees and find themselves unable to appreciate the situation of men in private practice. The older men among engineers are opposed to license laws as being in a sense degrading. The men who have architects' licenses are satisfied with things as they are. The men on salary who feel their positions secure are exceedingly conservative. The men in private practice are in a woeful minority. This state of affairs explains the reason why little has been done in the past to secure recognition for the engineer as a professional man. It shows why a code of ethics will be difficult to adopt.

I have said that 90% of engineers are on a salary. When one loses a position, he goes into private practice for a time pending the securing of another position. Few engineers in private practice went into it of their own accord but rather because they were laid off at a time when salaried positions were hard to secure.

Ninety per cent., it is safe to say, of the men educated as engineers knew positively nothing of conditions of employment and pay in the business when they registered at college for an engineering course. Many come from the homes of dissatisfied clerks and mechanics who felt their own lives to have been failures and desired for

their sons something better. Such boys studied engineering under the impression that it was a high-paying vocation and their home affiliations being with the class that extols the labor union, we have now a strong movement in the ranks of younger men to organize labor unions among men employed upon engineering work.

There is no real profession of engineering, at least not one so widespread as many would have us believe. There are a number of men with good business minds educated as engineers and to them engineering is commercial and not professional. Because of their keen business intellects they are apt to be dominant men when codes of ethics are discussed and they are generally, because of the advertising thus obtained, to be found heading committees and taking very prominent places in the councils of engineers. Many of them consist of the misplaced laborers, mechanics and clerks who were led to study engineering through misleading statements that it is an exceedingly remunerative occupation.

In the opinion of the writer, reform can only come by first teaching every high school boy and freshman in an engineering school as much about the vocation of the engineer as the followers of other vocations know about their future life work when they enter upon the study of it. The young men who then go to school will be well fitted to make a profession of the calling. The next thing to do is to eliminate from all committees having to do with the public relations of engineers all men employed on salary. This will leave the discussion of engineering ethics entirely to the private practitioners. Divide the men who call themselves engineers into two distinct classes, commercial engineers who are on salary or are engaged in manufacturing or contracting; and professional engineers who are in private practice. Until a true definition of an engineer is obtained, nothing can be done to remedy matters even after eliminating the many misplaced youths. More or less fanciful definitions are given of engineers but what is wanted is a positive definition of the kind of engineer who is in need of a good code of ethics.

ERNEST MCCULLOUGH.

Chicago, Mar. 5, 1913.

Tests of Large Brick Piers

Crushing tests of brick piers many times larger than any tested previously were made recently by the Bureau of Standards on the new 5000-ton testing machine erected in the Pittsburgh laboratory of the bureau. Heretofore it has been known, from a number of tests on small brick piers (12x12 or 12x16 in.), that the compressive strength of brickwork is much less than that of single brick. But how far this conclusion would apply to larger bodies of brickwork was not known, i.e., whether the larger piers would prove to be weaker, or stronger, or just as strong as small piers. The recent tests were made to give information on this question.

When brickwork is subjected to compressive load, the bricks first break in bending instead of in direct compression, and the result is to form long, vertical cracks in the mass, passing through the end joints of one course and the breaks in the brick of the next course. In this condition the brick mass is little more than an aggregation of slender vertical columns 4 in. square. Further loading produces a result which might be expected: the pier bursts or spalls. This result, which previously has been found for piers of small lateral dimensions, is now found also for large piers.

The crushing strength of the large piers tested is substantially the same as that of small piers (in pounds per square inch), and the manner of failure is the same.

The two piers tested were 48x48 in., by 12 ft. high. They were laid up directly on the table of the testing machine, and were tested at about one month (the second pier laid up after the first one was tested). One pier was laid in 1:1 cement mortar, the other in 1:3 lime mortar. The former was tested for modulus of elasticity when 16 days old (the test being confined to moderate loading so that the strength of the pier would not be affected); 15 days later, or at age 31 days, it was tested to failure. The lime-mortar pier was tested for modulus of elasticity and also to rupture at age of 25 days.

The cement-mortar pier failed at a load corresponding to 2917 lb. per sq.in.; an ordinary small-size pier of the same brick and the same mortar would be expected to show a strength of 3000 lb. per sq.in., in the Bureau's experience. This means that the strength of a large pier and a small pier is the same.

The lime-mortar pier failed at 757 lb. per sq.in. The same brick and mortar in a small-sized pier would be expected to fail at 900 lb. per sq.in., in the Bureau's experience.

The three views herewith show stages of the testing. Fig. 1 is the cement-mortar pier as laid up on the table of the machine and under test for modulus of elasticity. Fig. 2 shows this pier after failure. Fig. 3 shows the lime-mortar pier after failure. The more extensive split-

FIG. 1. BRICK PIER LAID IN CEMENT MORTAR, 48x48 IN. BY 12 FT. TEST FOR COMPRESSIBILITY IN 5000-TON TESTING MACHINE

FIG. 2. BRICK LAID IN CEMENT MORTAR FIG. 3. BRICK LAID IN LIME MORTAR

THE TWO 48x48-IN. BY 12-FT. BRICK PIERS AFTER TEST

ting apart apparent in Fig. 3 indicates that the lime mortar had much less bonding action than the cement mortar.

The moduli of elasticity of the two piers, at half the ultimate strength, were: Cement-mortar pier, 1,560,000 lb. per sq.in.; lime-mortar pier, 75,000 to 100,000 lb. per sq.in.

These tests were reported recently by J. E. Howard, Engineer-Physicist of the Bureau of Standards, in a paper before the National Brick Manufacturers' Association.

Licensing Engineers in New York

The movement to require engineers to pass examinations and receive licenses before practicing their profession in New York State is again being pushed in the New York Legislature. Readers of ENGINEERING NEWS will recall that a similar movement in New York two years ago was defeated in the Legislature through the efforts of prominent engineers who appeared before the committees in charge of the bill.

The bill now pending in the Legislature was introduced by Mr. Larrimer and is No. 1126 on the Assembly files. It is in general similar to the McGrath bill, introduced two years ago, and is understood, like it, to have been framed by and to have the backing of the Technical League of America. The Committee in charge of the bill will give a hearing upon it Apr. 1.

The pending bill is by its terms limited to the practice of civil engineering. It defines civil engineering as follows:

> A person practices civil engineering who holds himself out as able to do or undertakes to do work such as a civil engineer is called upon to do in the designing, construction and engineering inspection of public works, which require engineering experience and the same technical knowledge as engineering schools prescribe for graduation.

A brief summary of the provisions of the bill follows:

After Aug. 1, 1917, no person shall practice civil engineering as above defined in New York unless registered or licensed. The administration of the law is placed under the Board of Regents of the University of the State of New York. That body is to appoint a state board of engineering examiners, which shall register those qualified and shall hold examinations for the issue of licenses. To be either registered or admitted to examination, an engineer must pay a fee of $25. In order to take the examination, candidates must have graduated from an engineering course in an engineering school or correspondence school approved by the Regents. If a correspondence-school certificate is offered, candidates must also have had at least three years' experience in engineering work. In place of training in an engineering school, five years or more in practice may be accepted. The act provides means for revoking licenses of engineers who are guilty of any fraud or deceit.

The following persons may receive licenses without examination and for a $10 fee, by making application therefor before Aug. 1, 1917: Anyone actually engaged for three years in the practice of civil engineering; any

graduate from a four-year course in a reputable engineering school; any person who shall have been engaged in supervising, inspecting, designing or constructing public works of engineering character for three years or more. Engineer officers of the United States Army or Navy may practice their profession without license and without fees.

The bill is in the hands of the Assembly committee on public education, the members of which are Messrs. McKee, of Richmond, Wilmott of Kings, Larrimer of Kings, McMahon of Bronx, Fitzgerald of Erie, Doty of Orange, Grimme of Rockland, Lewis of New York, Hammer of Bronx, Schnirel of Ontario, Edwards of Broome, Webb of Putnam, Schaap of New York.

The committee gave a hearing on the bill in the State Capitol on Mar. 17. A dozen or more engineers resident in Albany, Troy and vicinity were present and opposed the bill on the ground that the engineering profession did not need or want such legislation. Mr. Eliot, representing the Technical League, spoke in favor of the bill and urged in its favor that many eminent engineers who last year opposed the bill were not present this year to oppose it because the bill had been redrawn to meet the objections then made. In reply it was said by those opposed that the engineering profession throughout the state had generally no knowledge that such a bill was pending.

It is stated, however, that the committee seemed to be prejudiced in favor of the bill. Engineers desirous of expressing their sentiments on the pending legislation can address letters to any member of the committee, or merely to the Assembly Committee on Public Education, Room 236, State Capitol, Albany, N. Y.

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Annual Convention of the American Railway Engineering Association

The 14th annual convention was held at the Congress Hotel, Chicago, Mar. 18 to 20, and there was the usual display of sustained interest in the proceedings which is characteristic of the meetings of this society. There was also a commendable promptitude in the attendance at the hours set for the meetings which is unfortunately a rare experience at conventions of this sort.

The address of the President, Mr. Churchill (N. & W. R.R.) reviewed some of the work and objects of the association, calling attention specially to its investigations regarding concrete and rails. The reports of the secretary and the treasurer showed a membership of 1066, and a balance of $10,745. The officers for 1913 are as follows: President, Edwin F. Wendt, Assistant Engineer, Pittsburgh & Lake Erie R.R.; Vice-Presidents, W. B. Storey, Jr., Vice-President, A., T. & S. F. Ry. System, and Robert Trimble, Chief Engineer of Maintenance of Way, Pennsylvania Lines (Northwest system); Treasurer, George H. Bremner, Engineer of Illinois District, C., B. & Q. R.R.; Secretary, E. H. Fritch, 900 South Michigan Ave., Chicago.

The 21 committees all had reports and all of these were presented at the meeting. They had been published in advance, and no time was wasted by reading reports in full. The chairman of each committee presented an outline of the report, and all conclusions were read, so that they might be discussed before being adopted. In this way the great majority of the time was occupied in active discussion of important matters.

The proceedings were disposed of in five long sessions, The annual reception was held the first evening, and was followed by an illustrated talk on the Panama Canal by Mr. McNab (Grand Trunk Ry.). The annual dinner was held the second evening, and on the third evening Prof W. K. Hatt (Purdue University) gave an illustrated talk on tests of rails rerolled into reinforcing bars for concrete.

In connection with the convention there was the annual exhibit of railway machinery and appliances, filling both the Coliseum and the adjacent Armory building. In order that this extensive and costly exhibit might receive a proper show of attention, the meeting was scheduled for four days, the last day being assigned entirely to visits to the exhibition. Referring again to the reports, we may note that most of them were comparatively short, and all were of a practical and definite character, dealing with certain assigned subjects. Three were of special importance in that they dealt with matters of a special character, vitally related to the economics of railway operation: (1) The report on economics of railway location discussed questions of grades and curvature in relation to train tonnage and operation; (2) the report on rails presented the results of exhaustive and valuable investigations as to rail manufacture and design; (3) the report on yards and terminals presented graphic systems for analyzing the train movements at passenger terminals and the train capacity of such terminals. We give below an outline of each of the reports, and abstracts of some of the reports will be published later.

Roadway

Ventilation of Subway Tunnels—This report presented a lengthy review of the principles of ventilation and the methods employed in ventilating numerous subway or rapid-transit tunnels in this country and abroad. The methods may be classed as follows: (1) Exhausting air at various points by fans; (2) forcing a current of air the whole length of the line by fans; (3) so called natural ventilation, by means of openings or "blow-holes"; (4) the piston action of trains. Although many subways are now provided with some system of ventilation by fans, by far the greater number still depend upon air currents set up without special mechanical aid.

Roadbed Pressures—The report advocated the appointment of a special committee (representing the roadway, ballast and track committees) to make experiments on the loads transmitted through the ballast and on the bearing power of roadbed materials. The experiments would necessitate considerable expenditure of time and money. The committee stated that the large number of rail failures, together with the constant increase in locomotive axle loads and the undoubted advent of the 100-ton car, make it imperative that some systematic study should be made of the roadbed as the foundation for the track. This report was accepted without discussion.

Ballast

The report recommended that as to physical tests, stone for ballast should be high in weight, hardness, toughness and compressive strength, and low in water absorp-

tion, wear and cementing qualities. An investigation made by the Pennsylvania R.R. shows that the total cost of ballasting with stone is $7911, $5708 and $5264 per mile for ¾-in., 1½-in. and 2½-in. stone respectively, while the cost of maintenance for four years is about $94.66, $84.94 and $82.13 per mile respectively. The report also discussed the methods of testing the quality of gravel for ballast, showing the results obtained from various gravels on different railways. An appendix described the method of cleaning stone ballast by the use of screens on the Baltimore & Ohio R.R. In line with the suggestion of the Roadway Committee (noted above), this committee presented an outline of a proposed test to determine the proper depth of ballast of different kinds to insure uniform distribution of loads upon the roadway; the cost is estimated at $3000 for three tests. The report was accepted without discussion.

Ties

In the report of 1912, the committee reported that it is not feasible to lay down fixed rules for the size and spacing of ties in relation to density of traffic, but this matter was then referred back to the committee with instructions to consider it from the point of view of the stresses to which the ties are subjected. The committee now reiterates its position as to the impracticability of the idea, due to the numerous factors (and the varying and indeterminate qualities of these factors) which affect the stresses in ties. The report contained also a detailed review of the various forms of steel tie-plates employed on various railways, and a continuation of the annual information relating to steel and concrete ties actually in use on steam and electric railways. The committee has not yet any definite information as to the relative effect of different designs of spikes and ties on the life of ties, but it is thought that experiments now being made will throw some light on the subject. The report was accepted without discussion.

Rails

This is one of the most important committees and it has done some very notable and original work in investigations of rail section and rail manufacture, besides preparing standard rail specifications which are now extensively used by the railways. The report this year was brief, but was accompanied by a number of appendices (nearly 400 pages) covering the committee's special investigation work, and its detailed statistics of rail failures. The principal feature of the report proper was in regard to rail sections. It stated that while some changes have been suggested, it is not yet time to consider them, as the A. R. A. sections, which are being used by a number of roads with good results, have been in service only two or three years. It is considered probable, however, that it may be found advisable to add a little metal to the upper and lower fillets.

The committee submitted also some changes in the wording of the specifications for carbon-steel rails (already adopted), and notes on the special investigations which have been carried on by Mr. Wickhorst, Engineer of Tests for the committee. As an indication of the importance of this work it may be noted that in the past three years the American Railway Association has appropriated $31,000 towards the expense. The report, being in the nature of a report of progress, was adopted without discussion, the proposed changes in the specifications being adopted also.

Track

The most important matter taken up by this committee is that of economics in track labor, this being the commencement of an exhaustive study of an important matter that has been largely neglected. The committee's work so far has been an investigation of the methods of educating and securing section foremen. The report presented also plans for main-line turnouts, a revised table of functions of the ten-chord spiral, and specifications for such track material as tie-plates, bolts, common spikes, screw spikes, spring nut-locks and anti-creepers. The specifications were discussed by paragraphs, and were adopted, with a few amendments. The turnout plans were adopted also. In view of recent disputes as to the proper design of crossovers in high-speed main tracks, Mr. Lindsay (N. Y. Central R.R.) suggested that the committee should take up the question of such crossovers, especially in regard to the curve at the switch angle, with switch rails of different lengths. In regard to the committee's investigation of the question of economics in track labor, Mr. Wendt (P. & L. E. R.R.) pointed out that wages represent an increasing proportion of the gross earnings, while the proportion represented by interest and dividends is on the decrease. He thought the consideration of economic questions one of the important developments in the work of the association.

Buildings

The report consisted mainly of a revision of former detailed information in regard to various roof coverings. There was also some information in regard to freight-house floors. While hard maple is generally preferable, beech and birch are sometimes used, and there is a good field for the introduction of gum (especially tupelo). This last is somewhat softer than maple, but it wears evenly and does not splinter much more than the former; there is a large supply of this timber in the South, and its use for floors is recommended.

The floor planks should be laid diagonally, except that if there is a pronounced amount of trucking in one direction they may be laid parallel to this direction, but if the traffic is heavy, a runway of steel plates may be built. Concrete has been used successfully where the wear is not too severe, and a concrete subfloor protected by a layer of asphalt will give good results, while asphalt blocks seem to have some advantages. The report was accepted without discussion.

Wooden Bridges and Trestles

Bridge Floors—Two important subjects are covered in this report: (1) Guard rails and guard timbers on bridge floors; (2) fire protection on bridge floors. In both cases, the matter consisted mainly of statements of practice on a number of individual railways, but as to the former, the following conclusions were adopted: (1) It is good practice to use outside guard timbers on open-floor bridges, the timbers being so constructed as to space the ties and hold them in place; (2) the guard timber and inner guard rail (when used) should be so spaced with reference to the track rail that a derailed truck will strike the inner guard rail before striking the timber; (4) the top of the guard rail should not be higher than

and not more than 1 in. below that of the track rail. Apparently no roads extend the outside timbers beyond the bridge (with the notable exception of the N. Y., O. & W. Ry.), this timber being considered as a tie spacer only and the inner guard rail being relied on to hold derailed trucks. Very few roads use rerailing devices, although these few include some important roads. For fire protection of floors on wooden bridges a covering of ballast or sheet metal is used, and sometimes both.

The discussion related particularly to the inside guard, and a recommendation in regard to the use and construction of such rails was referred back to the committee. This discussion showed a rather surprising amount of objection to guard rails, or rather to a requirement for their general use. This was partly on the score of doubtful efficiency, although we should consider their efficiency amply proved by experience; but the principal objection seemed to be the cost of equipping all bridges with these protective devices. The argument was made that they might as well be required on high embankments as at short bridge openings, but on the other hand it was shown that the length is not a factor, the more important consideration being the location of the structure.

Masonry

This committee presented only a brief report of progress, accompanied by an appendix on the disintegration of concrete and the corrosion of reinforcing metal. To prevent disintegration by the effects of sea water or water containing acids or alkalies, the concrete should be dense and impermeable, of good material and workmanship, and allowed to harden before being exposed to the action of the water. The use of special preparations is not considered necessary in good concrete. The report was accepted without discussion.

Signs, Fences and Crossings

The report presented a large amount of information as to fence posts of wood, metal and concrete, and submitted the conclusion that "concrete posts are practical, economical and a suitable substitute for wood." It presented also some further information as to securing fence wire that will resist corrosion and have good durability, but it recommended that this investigation be discontinued for the present, as certain experiments relative to it have been abandoned.

The third subject of the report was the construction of track in and across paved streets; this recommended 9-in. 141-lb. grooved rails on treated ties and a ballast bed 8 to 12 in. thick, this to be laid and rammed in 3-in. layers. In the discussion some changes were made to the specifications for concrete fence posts and to those for street crossings. As to the latter, there was a rather general opinion that the foundation was not sufficiently substantial, and that a heavier ballast bed or even a concrete base beneath the ballast might be necessary. Mr. Campbell (El P. & S. W. R.R.) said that he had found a concrete bed a great improvement in keeping the track in surface and mentioned cases where this is required by the city authorities.

Signals and Interlocking

This committee submitted a system of fundamental signal indications and aspects, and means of designating stop signals operated under automatic block system rules. The committee has considered also the effect of treated-wood ties and metal ties on track circuits. Its report states: (1) Each metal tie must be effectively insulated from the rail; (2) the effect of creosoted ties is not serious and with alternating current is negligible; (3) the effect of zinc-treated ties is more serious.* As to these last ties it appears that when they are used extensively track circuits a mile in length are rendered inoperative, but that circuits 2000 ft. long may be operated successfully even with 50% or more of treated ties. While the subject of signal indications and aspects has been under consideration for some years, this is the first time that the committee has been able to agree upon any one system. In presenting the report, Mr. Rudd (Pennsylvania R.R.) stated that these matters, as given in the report, have been adopted by the Railway Signal Association by a vote of 574 to 10. It was voted to adopt them and to refer them to the American Railway Association. The matter relating to track circuits was accepted as information.

Records and Accounts

This report showed the necessity of coöperation of different departments of railway service. It pointed out that the accounting department is as much interested as the maintenance-of-way department in the proper forms for time books, distribution sheets, authorizations of expenditures, etc., as these forms are the basis of the accounting system. Therefore, this association should seek the coöperation of the Railway Accounting Officers' Association in the development of a set of such forms. In the study of the economical management of store supplies the committee is already acting in coöperation with the Railway Storekeepers' Association. The committee also submitted a subdivision of the account for "roadway and track" in the Interstate Commerce Commission classification of operating expenses. This is desirable in order to analyze the expenses and secure uniformity in labor costs. The subdivision is as follows: (1) Track maintenance; (2) applying track materials; (3) cutting weeds and general clearing; (4) ditching and bank widening; (5) changing alignment and grades; (6) flood damage; (7) bank protection; (8) filling bridges; (9) other care of roadway and track (trackwalking, watching, etc.). The report was accepted without discussion.

Rules and Organization

This committee submitted a number of rules or instructions for maintenance-of-way employees, these being of a more specific character than the general rules which they supplement. They relate to such matters as gage on curves, superelevation and easement at ends of curves, switches, frog-guard rails, etc.; also rules for protecting unsafe and obstructed track and for the use of hand or motor cars in maintenance work.

The discussion occupied the greater part of one session, indicating that rules of this sort are considered of importance in relation to maintenance work. The committee's rule as to protecting track obstructed by repair work required stop signals to be displayed "not less than 20 telegraph poles" from the point of obstruction, but this was changed to read "in accordance with rule 99 of the American Railway Association," and it was suggested that in automatic block-signal territory the foreman

*See "Engineering News," Feb. 6, 1913.

should see that the signals governing the section to be obstructed are in the "stop" position, so that trains will enter the section under control. A new rule was adopted calling attention to the importance of the uniformity of gage, but the rule for widening gage on curves was adopted without dispute. This provides for a widening of ⅛-in. for each 2° over 8°, with a maximum widening of ¾-in. and a maximum gage (including wear) of 4 ft. 9½ in. At frogs on the inside of curves (which should be avoided where possible) the gage should be standard. Mr. Wendt (P. & L. E. R.R.) suggested that the committee should now undertake the study of the principles of organization of the maintenance-of-way department, which is an important subject needing more attention than has hitherto been given to it.

Water Service

Deep-Well Pumps—The report proper dealt with deep-well pumping apparatus, including reciprocating or plunger pumps, vertical-shaft centrifugal and propeller pumps, air-lifts and air-displacement pumps. In this last system, the air acts as a piston upon the surface of the water in a discharge chamber, depressing it and forcing it out through a pipe. It is shown that each has its field, but that the first named type is not adapted for operating scattered wells unless electric power is available; the efficiency may be increased by using double-acting and non-pulsating pumps. The vertical-shaft pumps are particularly adapted for handling water which contains a considerable amount of sand; the practical length of shaft does not usually exceed 200 ft. The air-lift is of low efficiency, but it can raise a larger amount of water from a small hole than any other form of apparatus; it can operate wells having sandy water, or those which are too small or too crooked for any other method. The air-lift, however, will not convey water horizontally, so that if the wells are at any distance from the tank a second pump is necessary, but this objection does not apply to the air-displacement pump. The report was supplemented by a paper on "Track Tanks" by G. W. Vaughan, N. Y. Central R.R. It was submitted as information, no conclusions being presented, and was accepted without discussion.

Yards and Terminals

Operating Capacity of Passenger Terminals—This report was of exceptional interest in dealing with the novel and important subject of the analysis of the operating capacity of passenger terminals. This analysis is made by the aid of diagrams or charts showing the train movements throughout the day, and showing at what times each platform track and approach track is occupied and clear, thus facilitating a study of changes for increasing the capacity. One of these chart systems has been worked out on the Belgian State Railways, and another on the Pennsylvania R.R., while the committee submitted a modified system. Sketch plans of terminals were given, with information as to the traffic and the facilities for handling passengers, baggage, express and mail. Another important subject included in the report was that of the mechanical handling of freight, this being supplementary to former reports. It included the telferage system, electric trucks, overhead traveling cranes with revolving horizontal booms under the trolleys, elevating and stacking conveyors, and freight-handling equipment for docks and steamship terminals.

The committee's recommendation as to limiting the curvature through switches on the approach or throat tracks of a passenger station to that of a No. 8 slip switch on tangent brought some protest from engineers who are using No. 6 and No. 7 slip switches, but on the other hand some thought the principle desirable, although it may not be possible to follow it in every case. One road has found it necessary to call the attention of its motive-power department to the derailment of engines on No. 7 turnouts which they were supposed to pass safely. This led to certain changes in the engines. For storage and loading tracks a minimum curvature of 6° was recommended, to avoid trouble in coupling cars. In two cases the committee recommended certain features as being "safe" but this word was eliminated, as it implied that any departure from the recommendation might be unsafe. In fact the suggestion was plainly made that there is too much talk of "safety first," since railways are built to handle business, and safety is only one of many factors in their efficient and economical operation. The report included some additional information in regard to hump or gravity switching yards, and its recommendations as to such yards were approved.

Iron and Steel Structures

Inspection—The committee submitted rules for the instruction of inspectors in mill, shop and field; while a minority report suggested that railways coöperate in establishing an inspection organization which would represent the railways in general. The idea was that this would be more satisfactory than to have the work done by individual railways or by private inspection bureaus, and it was suggested also that the rules submitted in the majority report would not apply to bureau inspection, which represents more tonnage than railway inspection. A progress report was made in regard to tests of built-up columns, which tests are to be made this year. The rules were read and discussed and were adopted with some few amendments. Mr. Lindsay (N. Y. Central R.R.) thought they should include instructions as to care in loading, unloading and storing the material, and as to looking for and rectifying members damaged in unloading or erecting. No reference was made to the minority report or to the relative merits of inspection by railway engineering departments and by private bureaus.

Economics of Railway Location

This was probably the most important of all the reports, dealing with matters which vitally affect the efficiency and economy of operation. One subject dealt with was the analysis of maintenance-of-way accounts in the general operating accounts. It was pointed out that while the majority of problems relative to this subject deal with main tracks, occasional problems introduce the consideration of side tracks, and it is therefore necessary to separate certain of the operating accounts for main-track cost and side-track cost. The committee presented an analysis of these accounts, with a formula for determining the annual charge per mile of main track for any account which embraces both main and side tracks. The report discussed also the relative damage to track by the locomotive and its train; figures from about 50 railways showed that the passenger locomotive does as much damage as the cars of its train, while the freight locomotive does about one-third as much damage as its cars. One of the conclusions presented was to the effect that one

ton of passenger train does more damage to track than one ton of freight train, and that the locomotive creates more disturbance than an equal weight of cars and contents. These conclusions were referred back to the committee, as they were admittedly only estimated and objection was made that as the increased damage due to passenger trains is on account of their speed, the running of high-speed freight trains would tend to lessen the difference.

Appended to the report was a paper on an extensive series of dynamometer tests made by the B. & O. R.R., and their relation to tonnage rating and train resistance. The committee submitted a scheme of varying grade compensation for curvature to meet different conditions. Also a statement that there is no absolute value for train resistance, and that for practical purposes freight-train resistance can be considered constant between 7 and 35 m.p.h. These were adopted, but some of the conclusions relating to formulas for tonnage rating were referred back to the committee for further consideration.

The discussion on this report occupied nearly an entire session. Mr. Sullivan (Can. Pac. Ry.) pointed out the value of the committee's work in analyzing operating conditions and costs in relation to physical conditions, and he considered that there should be greater coöperation between the engineering and operating departments in order to secure higher efficiency of operation. Mr. Tratman (ENGINEERING NEWS) thought the great importance of the report was in showing the advantages and actual economies which may be effected by introducing scientific refinements in location and maintenance. These are limited somewhat by the effects of weather, traffic and track labor in disturbing the actual grades, and he suggested that while the varying scale of compensation recommended by the committee is valuable as an ideal to be aimed at, it may be questioned whether these actual compensated grades would remain undisturbed for any long period of time under present conditions of track, traffic and maintenance work.

Wood Preservation

The committee reported that the use of water-gas creosote is limited but increasing, and that there is a growing sentiment in favor of mixing this with coal-tar creosote. The principal objection to the former seems to have been its unauthorized use as an adulterant of the latter, but its use is now increasing in legitimate or authorized mixtures. The report was accepted without discussion.

Electricity

ELECTROLYSIS—The principal part of the report of this committee dealt with the conditions of electrolysis and the means for its prevention. It included reference to investigations made and measures adopted for the Brooklyn Bridge, the Brooklyn and Chicago street railways, the Grand Central and the Pennsylvania terminals in New York, the Bergen tunnels of the D., L. & W. R.R. and Erie R. R., and the Baltimore terminal of the B. & O. R.R. This report was accompanied by an extensive bibliography. It was accepted as information, without discussion, after a brief explanation of its purport by the chairman of the committee, Mr. Kittredge (N. Y. Central R.R.).

Conservation of Natural Resources

The main feature of this report was its presentation of the work being done by the Canadian authorities for the protection and conservation and proper utilization of the timber, coal, peat, natural gas and oil resources. It is thought that large quantities of petroleum may be discovered in Alberta, which would be of great importance for the oil-burning locomotives on the coast lines of the Canadian Pacific Ry. and the Canadian oil-burning coasting steamers, all of which now use California oil. A rather surprising reactionary opinion in opposition to the modern principle of conservation was voiced by Mr. Downs (Ill. Cen. R.R.), but was answered very emphatically by Prof. Williams (Cornell College, Iowa), and others who showed the reckless waste and destruction of natural resources (instancing the increasing consumption and decreasing production of oil), and the responsibility of the railways in various directions. It was suggested that engineers and engineering associations certainly should lend their aid to a movement for the protection and proper use of the natural resources. This view was held almost unanimously, and the report was accepted and approved by a large vote.

Uniform General Contract Forms

This special committee presented the completed form of its construction contract, and asked to be discharged as it has now finished its work. The draft form has been submitted to all railways represented in the Association; of 70 replies, 35 approved it, 25 made some criticisms (generally as to wording or minor matters), 6 preferred their own forms, 3 did not criticize, and only one thought it impracticable to draw a uniform contract form. The contract was read in full and adopted, with a few amendments. It contains the usual clause giving to the engineer the power of deciding all questions and controversies which may arise between the company and the contractor, and this led to some discussion. As to the method of settling such disputes by arbitration, as has been suggested recently, Mr. McDonald (N., C. & St. L. R.R.) thought they could be settled in court quicker and at no greater expense. It was explained, however, that the committee was of opinion that the engineer should have this power while the work is under construction, but of course any case can be submitted to arbitration or to court action afterwards.

Grading of Lumber

One of the objects of this committee has been to secure the adoption of the rules drawn up jointly by the committee and the lumber associations. It reported that the Am. Ry. Eng. Association's rules for maintenance-of-way lumber for yellow pine, hardwoods and Douglas fir have been adopted by the respective lumber associations, but that those for white pine have not been accepted. It presented grading rules for cypress. The report was accepted without discussion.

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A Derailed Milk Train on the New York Central & Hudson River R.R., near Spuyten Duyvil, some eight miles north of the Grand Central Station, tied up all traffic on the main line of the railway to and from the Grand Central Terminal on Mar. 14. The accident occurred about 8 p.m., when a southbound milk train, approaching the city at a 20-mile rate, was derailed and the cars piled up across all four tracks. Although composed chiefly of milk cars the train is reported to have had a steel coal car near the middle, the hopper of which is supposed to have come loose and caught on the third rail, which carries the current for the electric locomotives. The third rail was torn up for some 500 ft., which necessitated the electric power being shut off for a considerable time. It was nearly two hours before the tracks were sufficiently cleared to permit a resumption of traffic.

The Commission Plan of City Government will be voted on in Jersey City, N. J., on Apr. 15.

An Explosion of Nitroglycerin destroyed one of the factories of the Burton Powder Co., at New Castle, Penn., on Mar. 17. The explosion was caused by fire and the 50 men employed at the works had time to escape before the explosion occurred.

A Rear Collision between two freight trains on the Union Pacific R.R., near Hearndon, Neb., on Mar. 14, caused the deaths of five trainmen. A blizzard was raging throughout the state at the time.

Another German Zeppelin Dirigible Balloon was wrecked by a windstorm on Mar. 20. The airship was known as "Substitute Z 1." After beating against a 50-mile gale for an hour and a half the crew succeeded in making a landing in safety near Karlsruhe. A second violent burst of wind caught the balloon at anchor and entirely demolished it.

A Gas Explosion at the canning factory of the Western Canada Cold Storage & Packing Plant, Medicine Hat, Alta., caused the deaths of five employees and serious injuries to a score or so more. The explosion is reported to have been caused by a small fire, which started while plumbers were repairing a leaky gas main. The building, a three-story structure, was entirely destroyed.

A Rear Collision between two fast express trains on the Union Pacific R.R., near Gothenberg, Neb., on Mar. 14, caused the deaths of four passengers and serious injuries to about 30 more. The accident occurred about 4 o'clock in the morning, during a blinding snowstorm. Train No. 4, known as the Atlantic Express, was reported to have been at rest, when train No. 12, known as the Colorado Special, crashed into the rear car, a Pullman sleeping car. This car and a coach ahead were partly demolished, and all the dead and most of the injured were in the sleeping car.

Severe Windstorms all through the Middle West between Mar. 20 and 22 caused widespread damage and a loss of life reported as nearly 100. The storm extended from Texas, through the Mississippi Valley, Alabama, Illinois, Ohio and western New York State. At Chicago, telegraph and telephone connection with outside cities was interrupted for several hours. Here the wind was accompanied by snow, sleet and rain, causing much damage to shipping. At Detroit, Mich., the wind is reported to have reached a velocity of 86 miles per hour, the highest ever attained in that city. In Indiana the damage done by the storm is estimated at $1,000,000.

A Severe Tornado, accompanying a cyclonic storm, swept across Nebraska and Iowa on Mar. 24. The greatest damage was done at Omaha, where telegraphic dispatches as we go to press report a death list of 152 and property damage estimated at $5,000,000. At Yutan, Neb., 16 persons were killed, at Council Bluffs, Iowa, 12 persons, at Terre Haute, Ind., 18 persons were killed and 250 injured. Other cities suffered less severely. The total number of lives lost is estimated in press dispatches at 239. Next to Omaha, the city of Terre Haute is believed to have suffered the most. The property loss here is estimated at $1,000,000.

The cyclonic storm extended from Modena, Utah, through Denver, Colo., Omaha, Neb., Milwaukee, Wis., across Lake Michigan to Sault Ste. Marie and through the Province of Quebec. This was the second severe storm in the Middle West within a week.

High Floods throughout Ohio and Indiana caused the loss of many lives in Dayton, Ohio, Peru, Ind., Delaware, Ohio, and many other cities and towns on Mar. 25. Many dams and bridges were destroyed and railway service in the flooded district has been practically adandoned.

The Mississippi River Levee Association has just been organized in Tennessee to disseminate information as to the practicability and feasability of the plans for levee construction and flood prevention in the Mississippi River Valley recommended by the Mississippi River Commission. A. S. Caldwell, a banker of Memphis, is president of the association, and Mr. John A. Fox is secretary-manager, with offices in the Scimitar Building.

Dock Construction in New York City, at an expense of $15,000,000, including outlays for land purchases and damages, was decided upon at a meeting of city officials on Mar. 17. Of the total amount it is stated that about $5,000,000 will be spent on a 1200-ft. pier and an adjacent half pier located at West 44th St. and the North River. There will also be two 800-ft piers near Desbrosses St., where the pier-head line is to be straightened; and a considerable amount of work will be done in Brooklyn.

PERSONALS

Dr. Willis L. Moore, Chief of the United States Weather Bureau, has announced that he will resign July 31.

Mr. Ernest Brown, an architect of Regina, Sask., has been appointed Deputy Minister of Public Works of the Province of Saskatchewan, Canada.

Mr. Alexander Dow, M. Am. Soc. M. E., General Manager of the Detroit (Mich.) Edison Co., has been elected President of the company and also of the Peninsula Electric Light Co.

Mr. H. W. Fenno, Resident Engineer of the Lake Shore & Michigan Southern Ry., at Erie, Penn., has been appointed Resident Engineer at Chicago, Ill., succeeding the late J. W. Crissey.

Mr. Marcus A. Dow has been appointed General Safety Agent of the New York Central & Hudson River R.R., with headquarters in New York City, succeeding Mr. George Bradshaw, resigned.

Mr. De Witt V. Moore, formerly associated with the W. C. Halstead Co., Contractors, of Indianapolis, Ind., has opened an office in the Merchants Bank Bldg., Indianapolis, as a consulting engineer and architect.

Capt. Julian L. Schley, Corps of Engineers, U. S. A., Assistant to the Engineer Commissioner of the District of Columbia, has been appointed Executive Officer of the new Public Utilities Commission of the district.

Mr. F. H. Fowler, Assoc. M. Am. Soc. C. E., formerly Hydro-electric Engineer, U. S. Forest Service, San Francisco, Calif., has been promoted to be District Engineer, succeeding Mr. Walter L. Thurber, resigned.

Mr. E. B. Dailey, Assistant Mechanical Engineer of the Union Pacific R.R., at Omaha, Neb., has been appointed Assistant to the Director of Purchases of the Southern Pacific Co., with headquarters in New York City.

Mr. Charles J. McGonigle, M. Am. Soc. C. E., formerly Contracting Manager of the Portland, Ore., office of Milliken Bros., Inc., has opened an office in the Chamber of Commerce Bldg., Portland, as a consulting structural engineer.

Mr. Charles Forrester has been appointed Superintendent of the Grand Trunk Ry., with headquarters at Stratford, Ont. Mr. Forrester began his railway work as a telegraph operator, and until recently was Trainmaster at Stratford.

Mr. John G. Greiner, formerly General Foreman of the shops of the Cincinnati, Hamilton & Dayton R.R., at Lima, Ohio, has been appointed Master Mechanic of the San Pedro, Los Angeles & Salt Lake R.R., with headquarters in Salt Lake City, Utah.

Mr. Almon L. Fales has resigned as Superintending Chemist of the Worcester, Mass., sewage-disposal works to become associated with Metcalf & Eddy, Consulting Engineers, Boston, Mass. Mr. Fales is succeeded at Worcester by Mr. Roy S. Lanphear, as noted in our issue of Jan. 9.

Mr. C. J. Morrison, M. Am. Soc. M. E., formerly associated with Suffern & Son, New York City, as Manager of the engineering department, has announced the formation of the firm of Froggatt, Morrison & Co., with offices at 149 Broadway, New York City. Mr. Morrison is Chief Engineer of the new firm.

Mr. Horace H. Esselstyn, M. Am. Soc's C. E. and M. E., has resigned his position as Engineer with Westinghouse, Church, Kerr & Co., to become Construction Engineer of the Detroit Edison Co., Detroit, Mich. Mr. Esselstyn has been associated with Westinghouse, Church, Kerr & Co., for the past 12 years.

Mr. David H. Blossom has resigned as City Engineer of Salt Lake City, Utah. Mr. Blossom is a graduate of Middlebury College, Vermont, and studied civil engineering at the Massachusetts Institute of Technology. For six years previous to his appointment as City Engineer, in 1911, he was Chief Engineer of the American Falls Canal & Power Co., at Salt Lake City.

Mr. J. B. Berry, M. Am. Soc. C. E., Chief Engineer of the Chicago, Rock Island & Pacific Ry., has been appointed Assistant to the President. Mr. Berry has been Chief Engineer of the Chicago, Rock Island & Pacific Ry. since 1905; previous to that time he had been Consulting Engineer of the St. Louis & San Francisco R.R. and Chief Engineer of the Union Pacific R.R. Mr. Berry is succeeded as Chief Engineer by Mr. Charles A. Morse, M. Am. Soc. C. E., as noted elsewhere.

Mr. E. E. Adams, Assoc. M. Am. Soc. C. E., Assistant to the Consulting Engineer of the Union Pacific System and Southern Pacific Co., has been appointed Consulting Engineer of